THE UNIVERSITY OF ALBERTA

A PRELIMINARY INVESTIGATION INTO THE SHEAR STRENGTH OF AN ASPHALT STABILIZED SAND IN TERMS OF EFFECTIVE STRESSES

by

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A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA

MAY 1967

UNIVERSITY OF ALBERTA

FACULTY OF GRADUATE STUDIES

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled A PRELIMINARY INVESTIGATION INTO THE SHEAR STRENGTH OF AN ASPHALT STABILIZED SAND IN TERMS OF EFFECTIVE STRESS submitted by Douglas Thomson Campbell in partial fulfilment of the requirements for the degree of Master of Science.

ABSTRACT

Asphalt stabilization of soils, although one of the oldest known methods of soil stabilization, is probably the least understood method of stabilization.

This preliminary investigation into the shear strength of an asphalt stabilized sand in terms of effective stresses assumed the applicability of the effective stress equations derived for saturated and partly saturated soils to asphalt stabilized soils. Triaxial test samples were prepared using a poorly graded fine sand and a 200-300 asphalt cement. The sand-asphalt mixes were attained using two methods - the foamed asphalt process and the wet hot mix process.

Results from the saturated test series showed a greater cohesion and a slightly lower angle of internal friction, in terms of effective stresses, for the foamed asphalt stabilized sand mix. The triaxial tests on the partly saturated samples were not successful in determining the shear strength in terms of effective stresses due to limitations of the testing equipment. Final results for this series were presented in terms of total stresses and showed the superior strength of the foamed asphalt sand mix. Microscopic examination revealed that the foamed asphalt method of preparation resulted in better distribution and a more uniform coating of asphalt throughout the sand particles. A postulation was put forward that the experimental method of determining values

of the factor X, in the equation $\sigma' = \sigma - u_a + X(u_a - u_w)$, for partly saturated soils may not be applicable to asphalt stabilized soils.

Major recommendations concerned the modifications and additions to the existing test equipment necessary for a successful testing program, and the necessity for a detailed physical chemical study of an asphalt stabilized soil.

ACKNOWLEDGEMENTS

Sincere appreciation is extended to the following:

Mr. K.O. Anderson, Associate Professor of Civil Engineering, for his constructive criticism and advice while carrying out this investigation;

The Royal Canadian Air Force under whose sponsorship the author was able to carry out his post-graduate program;

Dr. S. Thomson, Associate Professor of Civil Engineering, for his time and assistance in carrying out the microscopic examination of the sand-asphalt mixes;

Dr. E. W. Brooker, Associate Professor of Civil Engineering for loans of publications from his personal library;

My wife, Claudia, for typing drafts and for her moral support during the entire program.

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CHAPTER I

INTRODUCTION

1.1 General

The importance of stabilized soils has become evident as a result of the vastly increased construction program in highways and airfields during the past 20 or 30 years. This construction expansion has resulted in growing shortages of high quality gravel sources in many areas. This, coupled with the evergrowing need for roads and airfields, has necessitated the greater use of inferior materials in base and surface courses. To ensure the standards governing the quality of roads and airfields are not downgraded by the use of these inferior materials, a means must be available to upgrade the materials. A means commonly used to fulfill this requirement is soil stabilization. Soil stabilization may be defined as a treatment of natural soil so that after compaction it will provide a water resistant and stable structural layer of adequate load bearing qualities under the anticipated traffic and environmental conditions.

Soil stabilization treatments may be divided into three general groups (Puzinauskas and Kallas 1961).

(a) Materials which cement the soil particles. These are materials which are capable of reactions within themselves or with certain soil constituents in presence of water, producing strong interparticle bonds which can support high intensity loads. Typical

representatives of this group are Portland cement, limes and acidic phosphorous compounds. The soil stabilized with these materials may possess high initial strengths but due to the nature of the bonds formed, may lack desirable durability characteristics when exposed to such environmental conditions as drying-wetting or freezing-thawing cycles. Additionally, the quantities of such stabilizers required to produce adequate strengths are often economically prohibitive.

(b) Soil modifiers or conditioners. These chemical compounds, because of surface reactions with the soil minerals, change the soil texture and structure thereby altering the physical or engineering properties of the soil. Soil masses treated with these modifiers are often highly susceptible to climatic and environmental changes. Portland cement and lime at low concentrations, and calcium or sodium chlorides are representative stabilizers of this group.

(c) Waterproofing agents. The basic mechanism of soil stabilization by these agents is radically different from the two groups previously mentioned. These stabilizers coat individual soil particles to their agglomerates and hence hinder the penetration of water into the stabilized soil layer. Asphalts, certain resinous materials and coal tars are representative of this group.

Asphalt stabilization falls into the third group mentioned above. Stabilization of soils with asphalts is probably the oldest road-building process using admixtures as soil stabilizers. The role of the asphalt film in stabilizing a soil mass depends to a great extent on the properties of the soil. In the case of non-cohesive soils (such as sands and silts) the asphalt film serves a double purpose. First it waterproofs the soil mass and secondly it binds the soil particles together, contributing materially to the load bearing qualities of the stabilized layer. In the case of fine grained cohesive soils, waterproofing is the principle role of the asphalt film.

The main considerations usually employed in the method of stabilization to be used are structural strength and costs. Of the three above mentioned groups of soil stabilization, asphalt stabilization is the least expensive to perform. This method does not provide the structural strength of the other two groups, however it does provide a base course or surface course of greater flexibility and hence can undergo much greater strains before failure than can the mixtures provided by the other two groups.

Asphalt stabilized materials are generally used for (a) construction of bases for surface-treated roads with low traffic volume (b) some main roads having concrete or bituminous surfacing and (c) for water proofing subgrades to preserve their stability. Asphalt stabilization has been successfully used throughout much of the United States and in several provinces of Canada including Alberta and Manitoba. Although

advances in soil mechanics and asphalt technology have contributed to a better knowledge of soil-asphalt systems, asphalt stabilization remains largely an art form rather than a science. Knowledge of the basic properties of such systems remain relatively sparse. Many test methods and empirical formulae, although practiced for a long time and suitable for given local conditions, often confuse rather than clarify the problem.

1.2 Purpose of the Investigation and its Limitations

This investigation is part of a continuing series of investigations into asphalt stabilization of highway materials at the University of Alberta. Most of the past work within this research program has dealt with the strength relationships of a soil stabilized with cutback asphalt, emulsified asphalt and asphalt cement. Laplante (1963) investigated the shear strength relationships of a uniform fine grained sand stabilized with an emulsified asphalt and an asphalt cement, and also the effect upon strength of varying the gradation of sand in the sand-asphalt mixture. Haas (1963) dealt with the shear strength relationships of a uniform fine grained sand stabilized with a cutback asphalt and asphalt cement. Knowles (1962) compared the strength relationships of emulsified asphalt stabilized sands with sands stabilized with lime additives, while Pennell (1962) conducted a comparison of the shear strength of Portland cement stabilized sand with cutback asphalt stabilized sand. The sand used by Laplante, Haas and Pennell was from the same source and of the same gradation characteristics. The testing procedures of these three investigators were essentially the same and hence direct comparison of test results could be made. In all the in-

vestigations cited above, shear strength characteristics were determined using the triaxial compression test.

After carefully examining the previous work done on asphalt stabilized soils at the University of Alberta, it was decided to carry out a preliminary investigation into the effective stresses of an asphalt stabilized soil since the effective stresses would presumedly control the shear strength and volume change characteristics of an asphalt treated soil. The study was carried out utilizing the triaxial compression test, and the effective stress equations derived from Soil Mechanics research. It was hoped to obtain from this investigation the shear strength, pore pressure and volume change characteristics of an asphalt stabilized soil. An important phase of this investigation was also the testing techniques employed in the testing program.

The strength of sand-bituminous mixtures is affected by many variables including the quantity and type of asphalt, film thickness, type of aggregate, the size, shape and surface texture of sand particles, grain size distribution, the dry density and the amount of moulding water. Since all these variables cannot be taken into account in a single investigation, it was decided to make as many variables constant as possible. Only one type of soil was investigated, samples were prepared using the same asphalt content and were compacted to the same dry unit weight. Compaction of all samples was by kneading compaction and in order to obtain the same dry unit weights, compactive efforts were varied. Strain rates used in triaxial testing were the same for all triaxial tests and all tests were

performed at room temperature. Two types of asphalt stabilized soil were investigated, both using the same type of asphalt but each prepared by different methods.

No direct comparison of test results in this investigation to those carried out in previous investigations at the University of Alberta was possible due to the fact that a different soil and different testing techniques were employed. Also the main objective of measuring effective stresses was unlike those of previous investigations. Hence the conclusions drawn from this report are applicable only to the particular type of mixture used and the testing technique employed.

1.3 Organization of the Thesis

The second chapter deals with the principle of effective stress as applicable to soil mechanics. The effective stress equation is outlined for saturated soils and the present concept of the effective stress equation for partly saturated soils is presented.

The third chapter comprises the literature review and is divided into three sections (a) Soil Mechanics Literature Review (b) Asphalt Stabilized Soils Literature Review (c) Triaxial Compression Test. This division was considered necessary because these topics are in themselves separate entities, and are the main headings which must be taken into consideration in progressing with this investigation.

The fourth chapter is a classification and analysis of the materials used in the investigation.

The fifth chapter is devoted to an outline of the testing program and the procedures used. The testing techniques are given in some detail due to the fact that this was a major part of the investigation.

The sixth chapter summarizes the test results in tabular and graphical form and a detailed discussion is included.

The seventh chapter is a summary of conclusions and recommendations drawn from the test results and the literature review.

The Appendices follow a section on List of References and generally contain routine information on the classification of the sand, sample data and sample calculations as well as other minor information of insufficient importance to warrant its inclusion into the main body of the thesis.

CHAPTER II

PRINCIPLE OF EFFECTIVE STRESS AS APPLIED TO SOILS

2.1 General

A soil may be visualized as a compressible skeleton of soil particles enclosing voids, which in a saturated soil are filled with water and in a partly saturated soil are filled with water and air. Since shear stresses within a soil are carried only by the skeleton of the solid particles an adequate explanation of the shear strength characteristics of the soil can only be given in terms of the shear strength of the soil skeleton. The principle of effective stress offers such an explanation. The principle of effective stress is the assertion that all measurable effects of a change in stress, such as compression, distortion and a change of shearing resistance of a soil are exclusively due to changes in some function of the total stress and the pressure in the fluids within the voids of the soil. This function is known as the effective stress. Hence the effective stress acting on any plane in the soil may be considered as the stress carried by the solid particles in that plane and is equal to the total normal stress minus the pressure of the fluid in the pore space. The effective stress principle is also applicable to volume changes which occur within the soil but in this investigation the main emphasis is placed on the shear strength characteristics with respect to

effective stresses. The effective stress principle applied to saturated and partly saturated soils involves different considerations although the principle is essentially the same. Hence these two cases must be considered separately.

2.2 Effective Stress Principle Applied to Saturated Soils

Terzaghi (1936) first stated the effective stress principle in 1923 and was concerned with only a single fluid in the pore space of the soil. The effective stress principle for saturated soils may be summarized in two statements:

1. The effective stress σ' is equal to the total stress minus the pore pressure u_w.
2. The effective stress is the best available parameter to express certain aspects of soil behavior, notably compression and strength.

In studying the intergranular forces in a granular material it is necessary to consider a surface approximating to a plane but passing always through the pore space and the points of contact of the grains. FIGURE 1(a) illustrates this concept. Stresses and areas are then all considered as projected onto this plane.

Taking

σ = total stress on the plane

σ' = average intergranular force per unit area of the plane (= effective stress)

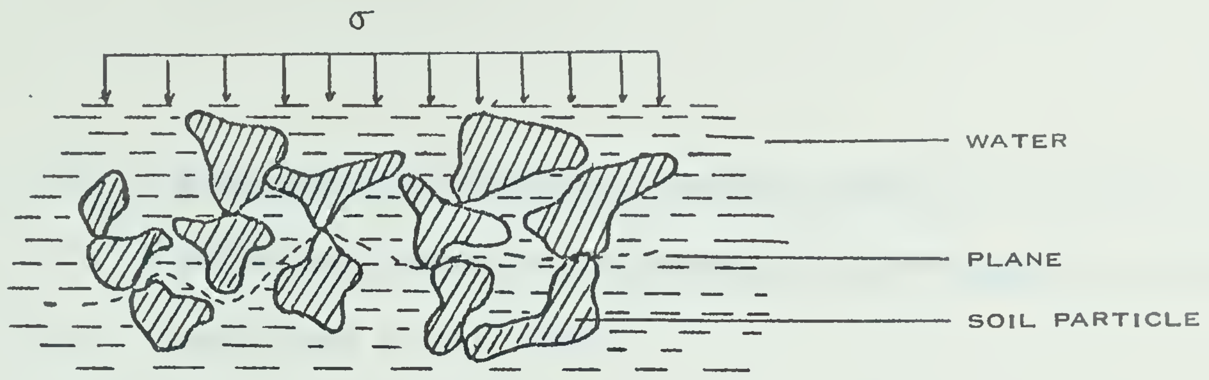


FIG. 1[A] SATURATED SOIL

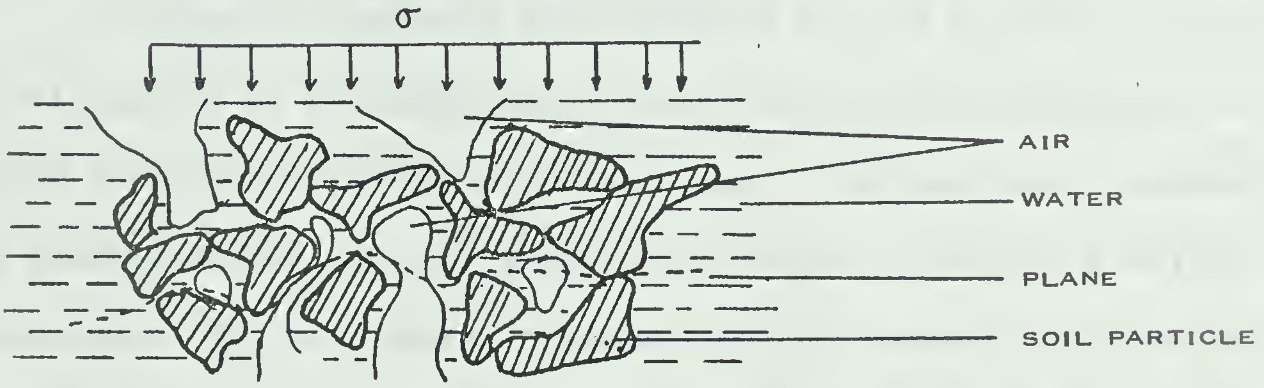


FIG. 1[B] PARTLY SATURATED SOIL

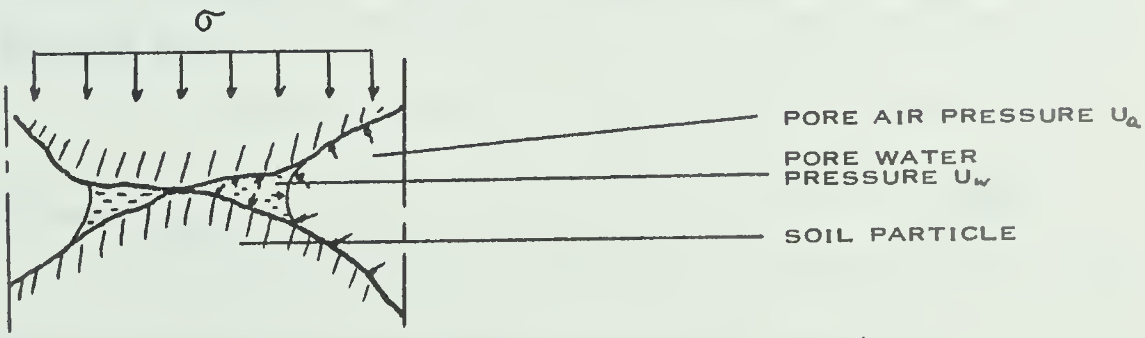


FIG. 1[C] PARTLY SATURATED SOIL WITH LOW DEGREE OF SATURATION

FIGURE I - EFFECTIVE STRESS PRINCIPLE OF SATURATED AND PARTLY SATURATED SOILS.

u_w = hydrostatic pressure in the pore water

a = effective contact area between the grains per unit area of the plane.

then $\sigma = \sigma' + (1 - a)\, u_w$

or $\sigma' = \sigma - (1 - a)\, u_w$ (1)

Terzaghi considered the effective area of contact (a) between the soil grains to be negligible. Hence the pore water pressure is considered to act over the whole of the plane. The pore water pressure u_w acts equally around the soil grains and changes in it do not effect the intergranular forces. Water is incompressible compared with the soil structure, therefore a change in the applied pressure is carried wholly by the pore water and the stresses in the soil structure are not changed unless drainage conditions permit a volume change.

By ignoring the area of contact a between the soil grains, Equation (1) is reduced to

$$\sigma' = \sigma - u_w \qquad (2)$$

The mechanical properties and hence the strength of a saturated soil are controlled by the effective stresses.

2.3 Effective Stress Principle Applied to Partly Saturated Soils

A partly saturated soil is a three phase system consisting of mineral grains, water and free air. Partly saturated soils may be either

naturally occurring or exist as compacted soils. Compacted soils are extensively used in engineering projects such as base courses for highways and airport runways.

A partly saturated soil is shown in FIGURE 1(b) and it is to be noted that the plane passing through the pore space and points of contact of the soil grains intersect both air and water phases. The partly saturated condition of the soil causes a "thirst" for additional water to be drawn into the soil. Because of this "thirst" for additional water, surface tensions exist in the water phase which causes the pressure in the water to be always less than the pressure in the air phase (Nash and Dixon 1960). The pressure in the water phase will now almost always be negative and the air in the continuous voids will be atmospheric. In the case of a soil with a low degree of saturation the pore water will be present as menisci in the vicinity of the interparticle contacts. This is shown in FIGURE 1(c). Here it is possible to consider that the pore water pressure acts over an area X per unit gross area of the soil and that the pore air pressure acts over an area (1-X). Because the pressures in the two phases present in the pore space differ, the effective stress formula given by Equation (2) must be modified to take into account the pressures in the pore air and the pore water.

Beginning in the mid 1950's, a number of investigators studied the aspects of partly saturated soils with regard to effective stresses. Bishop (1955), Hilf (1956), Aitchinson and Donald (1956), Croney et al (1958) and Jennings (1960) each presented a modified equation for the effective

stress principle applied to partly saturated soils. At the London conference on "Pore Pressure and Suction in Soils" held in 1960, it was concluded that the equations put forward by Bishop, Jennings, Croney et al and by Aitchinson were essentially the same. It was agreed at this conference that the equation for effective stress of partly saturated soils should take the form

$$\sigma' = \sigma - Xu_w - (1 - X)\, u_a \tag{3A}$$

or $$\sigma' = \sigma - u_a + X(u_a - u_w) \tag{3B}$$

where σ' = effective normal stress

σ = total normal stress

u_a = pore air pressure

u_w = pore water pressure

X = a factor representing a proportionate area of the soil over which the pore water pressure acts and is dependent on the degree of saturation, soil type and cycle of wetting and drying.

The equation put forward by Hilf (1956), based on Boyle's and Henry's laws, expresses the pore air pressure in a sealed soil specimen as a function of volume change. Hilf's equation is in the form of

$$\sigma = \sigma' + u_a + u_c$$

or

$$\sigma' = \sigma - u_a - u_c \qquad (4)$$

where u_c is the capillary pressure and is always negative. As pointed out by Langfelder (1964), if the soil is externally free of stress and the pore air pressure is zero (atmospheric) the capillary pressure u_c is equal to the negative pore water pressure u_w and Hilf's equation would degenerate to $\sigma' = -u_w$ which is identical in form to Terzaghi's equation for saturated soils. It is generally agreed by most investigators that the term $(-u_w)$ must be modified by a factor X in order to evaluate the effective stress and that the mathematical statement of the principle of effective stresses in partly saturated soils generally assumes the form shown in Equation (3).

In Equation (3) X is a variable factor which represents the proportion of suction or the capillary pressure $(u_a - u_w)$ that contributes to the effective stress of the soil. When the soil is fully saturated, the pore air pressure u_a is zero and $X = 1$. When the soil is dry, the pore water pressure is zero and $X = 0$. The intermediate values of X must, at the present time, be determined experimentally.

CHAPTER III

LITERATURE REVIEW

3.1 General

In order to clearly understand the numerous complexities involved in this particular investigation of an asphalt stabilized soil it is considered advisable to divide the literature review into three distinct sections embracing the three main facets of the investigation. The first section is a review of the literature in Soil Mechanics pertaining to the validity of the effective stress equations and to the pore pressures in a compacted soil. The second section reviews the literature pertaining to asphalt stabilized soils and considers the effects of the addition of asphalt to the soil on the stability of the soil in terms of shear strength. The third section reviews the literature of Soil Mechanics and Asphalt Technology pertaining to the triaxial compression test, its uses, methods, limitations and sources of error.

A. Soil Mechanics Literature Review

3.2 Validity of the Effective Stress Equation as Applied to Saturated Soils

The principle of effective stress as put forward by Terzaghi utilizing the equation $\sigma' = \sigma - u$ applies only to saturated soils. The basic assumption of the equation is that the pressure in the pore fluid operates over the whole area of the plane depicted in FIGURE 1(a), Section

2.1, an assumption which corresponds to point contact between particles of the soil mass. This assumption was recognized by Terzaghi as being a possible error in the equation but the areas of contact between soil particles were considered to be so small that they could be safely neglected without error.

Skempton (1960) in analyzing results from effective stress tests on various soils, rocks and concrete concluded that the equation as applied to soils holds true to a sufficiently high degree of accuracy for engineering purposes. Skempton (1960) reported that other investigators including L. Rendulic, D.W. Taylor and A.W. Bishop have verified that Terzaghi's equation for effective stress in saturated sands and clays involves no significant error within the range of pore pressures encountered in practice.

Bishop (1960) points out that the equation holds equally true if the pore space of a soil is completely filled with any other liquid or gas although the soil properties such as internal friction, cohesion and compressibility may be modified, particularly in fine grained cohesive soils.

The validity of the effective stress equation as applied to saturated soils is apparently accepted by most soil mechanics authorities. No opposition to the validity of the equation as applied to saturated soils could be found in the Soil Mechanics literature review.

3.3 Validity of Effective Stress Equation as Applied to Partly Saturated Soils

Whereas the effective stress equation as applied to saturated

soils has received world recognition and is widely used in both practice and research, the effective stress equation as applied to partly saturated soils has not yet been widely received and to the present time has only been used in research with the view to proving its validity. The equation, presented in Section 2.2, is in the form

$$\sigma' = \sigma - u_a + X\,(u_a - u_w) \tag{3B}$$

It is generally agreed by most authors that the principle of effective stress is applicable to partly saturated soils and that the equation is probably in the form shown above. For example, Lambe (1960) endorsed the effective stress principle in partly saturated soils and used a physical-chemical approach in confirming the validity of Equation (3B).

Bishop and Donald (1961), Bishop and Blight (1963) performed triaxial tests on a partly saturated silt and a partly saturated clay with a view to verifying experimentally the validity of Equation (3B). It was concluded that Equation (3B) is correct for the soils tested and that the behavior of the soils was independent of the absolute values of σ , u_w and u_a , but dependent only on the absolute value of σ'.

Jennings and Burland (1962), from experimental testing, came to the conclusion that σ' does not control the behavior of the majority of partly saturated soils but rather functions of the values of σ , u_w and u_a . They suggested that the equation actually defines the intergranular stress, which does not necessarily control soil behavior. Also

from their experimental work, Jennings and Burland concluded that there is a critical degree of saturation below which Equation (3B) does not apply. Suggested lower limiting values of critical degrees of saturation are for coarse granular materials, 20 per cent; silts, in the order of 40 to 50 per cent and for clayey soils upward of 85 per cent.

A major problem in using Equation (3B) is the determination of values for X . The value of X for saturated soils is unity and for completely dry soils is zero. Intermediate values depend on the degree of saturation, structure and cycle of wetting and drying of the soil. At the present time X for partly saturated soils must be determined experimentally and this requires that the true effective stress in the soil be known beforehand. The values of X can only be inferred by comparing the effective stress components $(\sigma - u_a)$ and $(u_a - u_w)$ with the known effective stress in the same soil when saturated (Bishop et al 1960). Jennings and Burland (1962) pointed out that when certain partly saturated soils under loading are inundated, they undergo a spontaneous compression or collapse volume change which makes it difficult to establish experimental values of X . Bishop and Blight (1963) reached the conclusion that effective stress parameters established for conditions of shear failure are insensitive to the effect of inundation. Burland (1964) has shown that regardless of possible collapse on inundation, saturated and partly saturated samples of a given soil with the same strength, reach identical void ratios at failure.

It appears that there is general agreement concerning the

validity of Equation (3B). This equation has been experimentally proven, but only for certain soils at particular degrees of saturation. Much investigation remains to be done, particularly in the evaluation of X, before this equation can be used in practical applications and before it can receive the widespread acceptance of Terzaghi's equation for saturated soils.

3.4 Pore Pressures in Compacted Soils

A compacted soil is generally a partly saturated soil and is composed of mineral grains in contact with each other surrounded by the pore fluid. The solid particles constitute the soil skeleton which determines the mechanical properties of the mass such as volume change under load and shearing strength.

Both air and water are present in the voids of a compacted soil; air being distributed as occulted air bubbles and air filled voids, water being in the form of adsorbed water films and free water. Aitchinson (1956), Yoshimi and Osterberg (1963) have concluded the greater part of soil air in soils compacted dry of optimum moisture content is contained in continuous air voids open to the external atmosphere.

The pressure condition at the air - water interface may be defined in terms of the comparable conditions of an idealized capillary tube (Aitchinson 1956). This pressure condition is referred to in this investigation as the capillary pressure u_c and is sometimes referred to as a measure of "thirst" of the soil for additional water.

$$u_c = \frac{2T \cos \theta}{r} \tag{5}$$

where T = surface tension per unit length of boundary

r = radius of curvature of the capillary tube formed by the soil particles

θ = wetting angle = 0 for soil-water-air

Moisture content, size of soil particles and state of packing of a partly saturated soil affect the capillary pressure u_c through their effect on the curvature of menisci r.

Due to the presence of surface tension in the water phase, the pore water pressure is negative while the air is at, or near, atmospheric pressure. The capillary pressure is negative and is equivalent to the term $(u_a - u_w)$ in Equation (3B). Thus the capillary pressure aids the effective stress of a compacted soil.

The pressure u_a in the air voids of a compacted soil which has been compressed or sheared without permitting the escape of pore fluids may be measured directly (Bishop 1960) or may be calculated by combining Boyle's law of compressibility of air with Henry's law of solubility of air in water. The resulting equation (Hilf 1956) is in the form

$$u_a = \frac{P_a \, \Delta V}{Va_o + hV_w - \Delta V} \tag{6}$$

where P_a = atmospheric pressure

h = .02 = coefficient of air solubility in water by volume according to Henry's law

ΔV = change in volume of entire soil specimen, per cent of initial volume

Va_o = initial volume of free air, per cent of initial volume

V_w = volume of water, per cent of initial volume

The pore water pressure u_w developed as a result of compression or shearing may be measured directly.

3.5 Evaluation of the Shear Strength of Compacted Soils

The shear strength of a soil is a measure of the soil stability with respect to shear stresses set up on the soil skeleton under conditions of loading. The pore fluids in the soil do not resist shear stresses and hence the mechanical properties of the soil are controlled entirely by the stresses on the soil skeleton.

The shear strength of soils was originally divided by Coulomb in 1776 into two parts, frictional strength and strength due to cohesion, and expressed by an equation of the form

$$s = c + \sigma \tan \phi \qquad (7A)$$

where s = shear strength of the soil

c = cohesion

σ = total normal stress on the failure plane

$\emptyset$ = angle of shearing resistance

With the introduction of the principle of effective stress by Terzaghi in 1923, it has become evident that in order to define the true shear strength of a soil, Equation (7A) must be modified in terms of effective stresses. For saturated soils the form of the modified equation is

$$s = c' + (\sigma - u_w) \tan \phi' \tag{7B}$$

where c' = cohesion in terms of effective stress

$\emptyset'$ = angle of shear resistance in terms of effective stress

u_w = pore water pressure

For partly saturated soils, Equation (7B) must be further modified to the form

$$s = c' + [\sigma - u_a + X\,(u_a - u_w)] \tan \phi' \tag{7C}$$

where u_a = pore air pressure

X = a factor dependent on the degree of saturation, soil type and cycle of wetting and drying.

The physical component of shear strength $\emptyset'$ is attributed to the frictional resistance and interlocking between particles. These physical factors are proportional to the effective normal stress on the failure plane and are of significance primarily between the granular particles of

the soil. The cohesion c' is taken to mean that part of the soil strength that is present independently of any applied pressures, either mechanical or capillary, and would remain if all applied pressures were removed. The shear strength parameters c' and ϕ' are not, for a given soil, constant for different loading conditions, stress history or testing conditions.

In the laboratory, the triaxial test is a means of determining the shear strength of soil samples. Test results for each soil sample are presented in the form of a Mohr stress circle. The common tangent to the Mohr stress circles for various samples under different confining pressures results in the Mohr failure envelope as shown in FIGURE 2(a) and this envelope defines the shear strength parameters c' and ϕ'. An alternative method to presenting triaxial test results is the use of the modified Mohr failure envelope in which individual test results are presented as points. The resulting failure envelope as shown in FIGURE 2(b) is related to the common Mohr failure envelope by the equations (Hvorslev 1960)

$$\sin \phi' = \tan \beta' \tag{8A}$$

$$c' = \frac{d'}{\cos \phi'} \tag{8B}$$

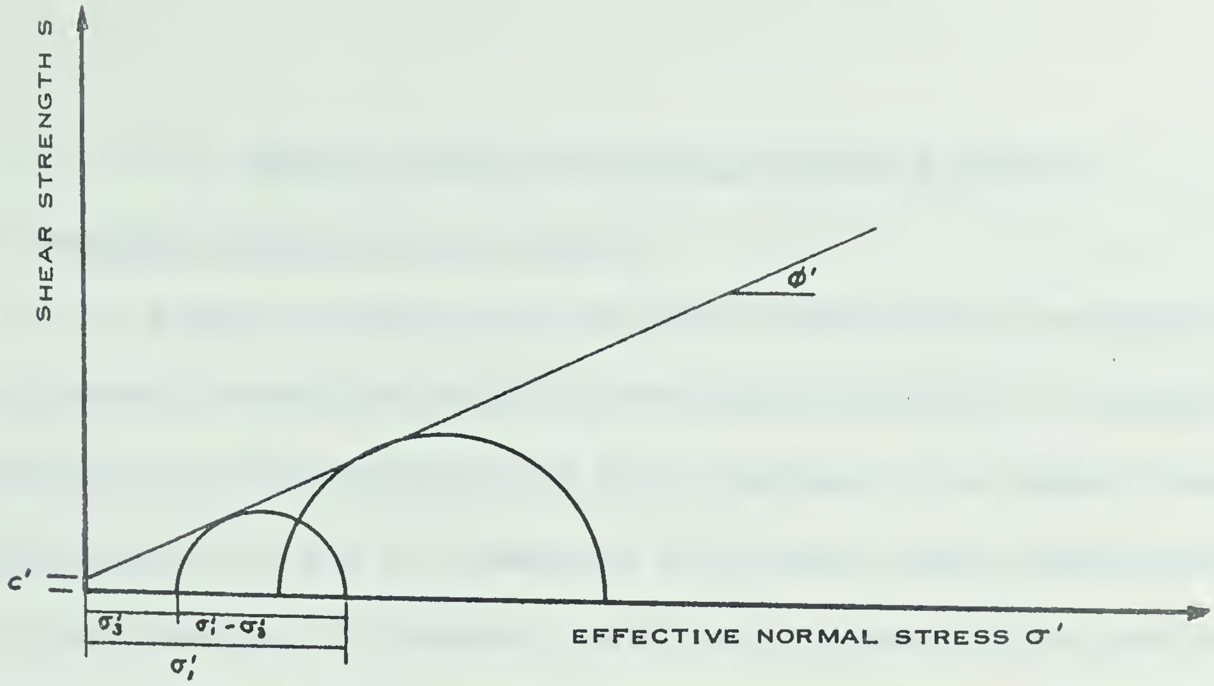


FIG 2[A] MOHR FAILURE ENVELOPE

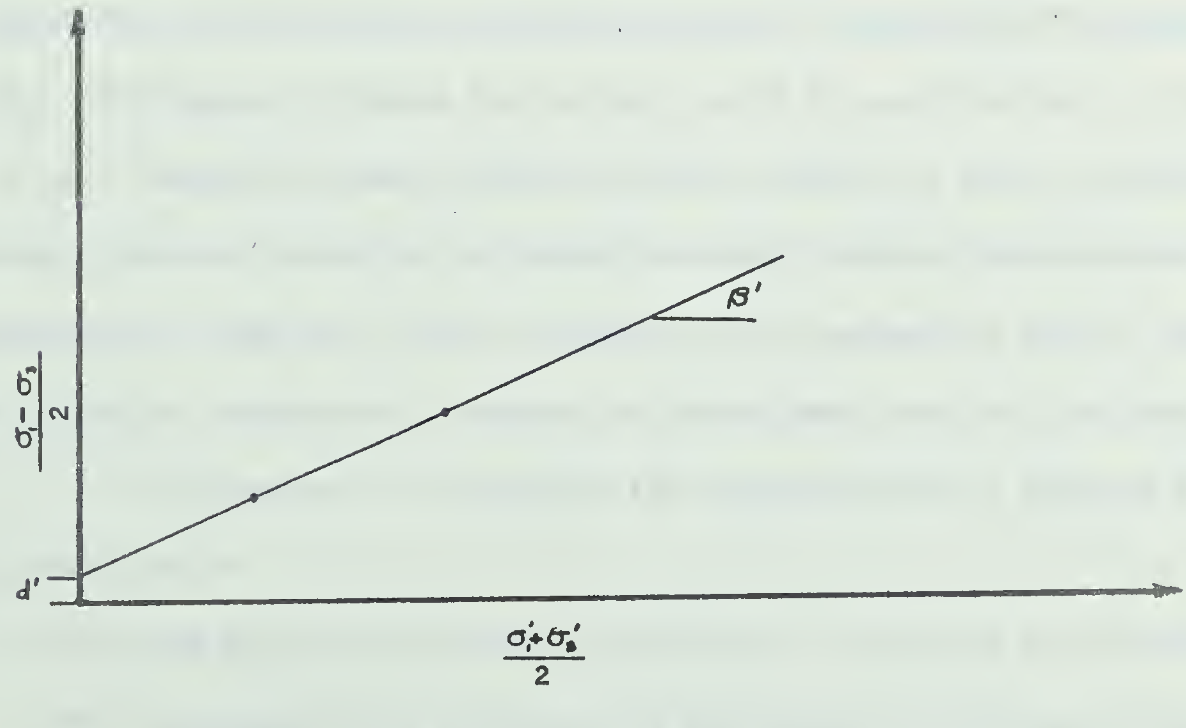


FIG 2[B] MODIFIED MOHR FAILURE ENVELOPE

FIGURE 2 - SHEAR STRENGTH OF COMPACTED SOILS

B. Asphalt Stabilized Soils Literature Review

3.6 Asphalt Stabilization of Soils

Asphalt stabilization of soils refers to the treatment of the soil where a controlled amount of bituminous material is thoroughly mixed with the soil. The objective of this treatment is to impart some favorable characteristic to the soil which in its present state does not meet stability requirements. If properly done, the treatment of the soil with the asphalt results in an increased resistance to deformation and thus to shearing action. A degree of waterproofing is also attained which aids in maintaining the improved strength characteristics of the soil-asphalt mixture.

The effectiveness of asphalt as a stabilizing material is limited by the difficulty of distributing it uniformly throughout the soil. The degree to which the asphalt will be satisfactorily distributed in a soil depends on many factors such as amount of water present during mixing, type and duration of mixing process, mixing temperatures, physical properties of the soil and the asphalt, the compactive effort used and the climatic conditions to which the stabilized mass will be subjected.

Two methods of attaining the distribution of asphalt throughout the soil are by

(1) modifying the asphalt physically to increase its fluidity.

(2) decreasing the viscosity of the asphalt by means of heating or foaming.

Method (1) involves the use of cutback asphalts or emulsified asphalts.

Cutback asphalts are mixtures of asphalt and volatile solvents. After mixing with the soil is accomplished, the mixture, during placing and compacting, must be aerated to allow the volatiles and water to evaporate thus permitting the asphalt to spread out and adhere to the soil particle surfaces. Turnbull and Foster (1952) point out that stabilization with a cutback asphalt, in practice, is dependent to a large extent on the weather as fairly long periods of dry weather are necessary for success.

Emulsified asphalts are mixtures of asphalt and water which produce an emulsion. After mixing an emulsified asphalt-soil mixture, the moisture released when the emulsion breaks must be reduced by aeration before the bitumen will coat and adhere to the particles. Until this is accomplished the bitumen floats in the mix resulting in very low stability. Hence stabilization using emulsified asphalt in practice is also dependent on dry weather conditions.

The second method of attaining asphalt distribution mentioned above involves asphalt which has been heated or foamed. If heat is used to liquefy a normally semi solid asphalt then the soil aggregate must be dried and heated which practically limits the use of this method to gravels and sands which can be run through an aggregate drier. The temperature of the soil and the asphalt must be approximately the same during mixing. Sharpe (1959) reports this method has been used on projects in Manitoba and Minnesota but the cost was high compared to other methods of asphalt stabilization.

When an asphalt is foamed by injecting steam into the asphalt,

it acquires unusual properties. With foaming, the asphalt's volume increases, viscosity decreases and the "wetting" action of the asphalt is increased during the mixing process due to the low surface tension the foamed asphalt possesses. When the foam bubble collapses in coating a mineral particle the entrapped moisture vapor escapes and, according to Csanyi and Nady (1958), the residual asphalt rapidly regains its original properties. The foamed asphalt process (Csanyi 1957) was devised by L.H. Csanyi during the period 1954 to 1956. Haas (1963) and Laplante (1963) have given a detailed review of the literature on foamed asphalt and these references are recommended for a more complete discussion on the mechanics of foamed asphalt stabilization. The foamed asphalt process of soil stabilization apparently excels over the other processes mentioned above in that the foamed asphalt can coat fine grained soil particles in a cold, damp condition.

3.7 Aspects of Some of the Components of an Asphalt Stabilized Soil

An asphalt stabilized soil consists of soil particles, asphalt, water and air. The theoretical structure of an asphalt stabilized soil consists of soil particles bearing on their surfaces asphalt films of varying thicknesses and degrees of coverage. These coated soil bodies, when compacted, result in a mass relatively stable in the presence of water.

The presence of water in an asphalt stabilized soil is necessary during the mixing and compaction processes. Water lubricates the asphalt and promotes its spread over the soil particles. It has been observed (Benson and Becker 1942) that the contact angle between the asphalt and

the soil particles is decreased with an increase in water thus permitting easier spreading of the asphalt throughout the mix. The presence of water in the mixture is necessary also to ensure proper compaction, however too much water leads to compaction and curing difficulties. The proper water content to use for good distribution of asphalt and compaction requirements is not always the best water content for desirable stability, but as suggested by Katti et al (1959) the optimum moisture content used to obtain maximum dry unit weight of the soil alone is probably the proper moisture content to be used in a soil-asphalt mix.

To attain optimum conditions in an asphalt stabilized soil it is necessary that the asphalt film be spread uniformily and thinly on all exposed surfaces of the soil particles. A film that is too thick merely lubricates the soil particles whereas a film that is too thin does not result in desired cohesion. The viscosity, surface tension and internal cohesion of the asphalt are probably the most important factors that affect the forming of thin films on soil particles (Csanyi and Fung 1955). When viscosity increases, surface tension and internal cohesion increase and vice-versa. As viscosity increases, cohesion increases and the spread of the asphalt film is retarded resulting in the formation of thicker films. Thinner films tend to form on aggregates when the binder is more widely distributed during its introduction into the mix.

3.8 Capillary Pressures in an Asphalt-Stabilized Soil

When asphalt is used for stabilization an additional phase with properties different from those of soil, water and air is introduced.

The presence of asphalt in the ideal form of a continuous membrane surrounding individual soil particles results in what is known as a hydrophobic or "water hating" mass. The mass has no "thirst" for additional water and in fact repels water which sets up a capillary pressure which is positive (Fossberg 1964). In the equation

$$u_c = \frac{2T \cos \theta}{r} \qquad (5)$$

for the ideal case, the angle of wetting between asphalt and water is greater than 90^o. The ideal case is seldom achieved in asphalt stabilized soils due to the method of mixing and the resulting discontinuous films. However the presence of asphalt induces a positive angle of wetting as opposed to a negligible angle of wetting for untreated soils, hence causing a reduction in capillary pressures. This phenomenon is also influenced by the fact that the asphalt fills and plugs some of the capillaries in the system. McLeod (1939) has shown, by laboratory testing, the effect of increasing asphalt content on the reduction of capillary pressures. Michaels and Puzinaskas (1958) observed that both the rate of imbibition and the ultimate water saturation are significantly less than those in an untreated compacted soil.

Besides capillary action in the water phase, Douglas and Tons (1961), and Sommer (1957) assert that there are capillary pressures in the asphalt phase of an asphalt-sand mix. Douglas and Tons made this assertion concerning a voidless mix of aggregate and asphalt and Sommer's assertion

was made with regard to a mix prepared by the impact process which presumedly results in the formation of thin continuous films. That capillary pressures exist in the asphalt phase was based on the Law of Poiseuille which, in mathematical form, states

$$v = \frac{\P p a^4}{8\,l\,\eta} \qquad (8)$$

where v = volume of liquid flowing through the capillary per second

p = pressure head over length l

l = length of capillary

η = absolute viscosity

a = capillary radius

Hence the amount of flow or flow potential in capillaries is proportional to the fourth power of the capillary diameter and is inversely proportional to the absolute viscosity of the liquid.

The concept of capillary pressures in the asphalt phase of an asphalt film has not received wide recognition. The only references that could be found on this subject are those cited above. Haas (1963) postulated that the existence of capillary pressures in the asphalt binder offers an explanation for the high shear strength characteristics of cured foamed asphalt stabilized samples.

3.9 Factors Affecting the Shear Strength of an Asphalt Stabilized Soil

The resistance of an asphalt stabilized soil to shear failure is dependent on the intergranular friction as measured by the angle of internal friction and the internal resistance of the asphalt film as measured by the cohesion.

Internal friction is influenced by the dry density of the mix and by properties of the aggregate such as hardness, surface texture and shape. The dry density of a mix is a function of aggregate gradation, asphalt viscosity and the asphalt and water content at which the mix will possess a maximum dry density for a given compactive effort. For a given aggregate, moisture content and asphalt content, the angle of internal friction varies with the density of the soil mass.

Cohesion is dependent on the asphalt films and is developed through the adhesion of the asphalt film to the soil particles and through the internal resistance of the asphalt film to deformation. Mack (1965A) states that thin films of asphalt surrounding soil particles behave more like solids due to physical chemical effects which result in increased viscosity and high elastic strength. Mack (1965B) and Fossberg (1964) point out that the internal resistance to deformation (or viscosity) of an asphalt film is greatly influenced by the interplay of surface tension forces and the respective wetting angles between the phases, which in turn results in the formation of interfacial tensions between the phases. Depending on the phases present and the respective wetting angles, the viscosity of the asphalt film and the capillary pressure in the water

phase varies with the magnitude of the resulting interfacial tensions. Some typical values for interfacial tension between the phases of an asphalt stabilized soil are given in TABLE I.

TABLE I

TYPICAL VALUES FOR INTERFACIAL TENSIONS BETWEEN PHASES OF AN ASPHALT STABILIZED SOIL (After Fossberg 1964)

Interface	Interfacial Tension $ergs/cm^2$ or dynes/cm
Water-air	72
Asphalt-air	26
Asphalt-water	30 ± 5
Solid-air	76
Solid-asphalt	Very large variation

The presence of interfacial tensions between phases appears analogous to the idea of capillary pressures in the asphalt phase in that they both offer an explanation to the increased resistance to flow of an asphalt film. Interfacial tensions however have been experimentally evaluated (Mack 1965B, Rice 1958) whereas capillary pressures in the asphalt film of an asphalt stabilized soil have not been measured. It appears, at the present time, that the presence of interfacial tensions between phases may offer a rational explanation for increased resistance

to flow in asphalt films.

The magnitude of the cohesion developed in an asphalt stabilized soil is influenced by the amount of asphalt in the mix, the thickness and viscosity of the asphalt film and the temperature of the mix. The elastic strength of an asphalt film increases with increasing thickness to a maximum and then decreases. Experimental evidence reported by Mack (1965B) shows that flow in an asphalt film takes place at thicknesses in the order of .01 centimeter and above. Mack also reports that asphalt films are thinnest at points of contact between soil particles and that the film strength depends upon the strength of the films at these points.

3.10 The Effective Stress Principle Applied to Asphalt Stabilized Soils

The literature available on the effective stress principle as applied to asphalt stabilized soils is limited and only one paper could be found on the subject. Fossberg (1964) stated that the effective stresses in an asphalt soil are largely governed by the viscosity of the asphalt and the interplay of interfacial tension forces with the viscosity. This makes the shear strength analysis based on effective stresses a very complex problem. Fossberg proposes that the effects of interfacial tensions between the various phases on viscosity and the cohesion due to viscosity be considered part of the bulk soil properties. On this basis, he suggests "the principle of effective stress can be employed, bearing in mind the effect of asphalt on the properties of the air-water phase." Fossberg investigated the effective stresses of asphalt stabilized soil samples in a saturated condition but did not attempt to measure the effective stresses in a partly

saturated condition.

If the interfacial tensions between phases are of sufficient magnitude so as to greatly affect the viscosity of the asphalt films, then it is postulated that the method outlined in Section 3.12 for determining values of the factor X in Equation 3 may not be applicable to asphalt stabilized soils. A sufficient change in the viscosity of the asphalt films would most likely result in saturated and partly saturated samples having different strength parameters in terms of effective stresses. The effective stress line obtained for saturated samples could not be assumed to represent the true effective stress envelope for partly saturated asphalt stabilized samples. That the effective stress line obtained for saturated soils represents the true effective stress line for partly saturated soils is the basic assumption in the experimental determination of X values.

In a preliminary investigation of the effective stresses of an asphalt stabilized soil, it would appear logical to assume that the effective stress equations are also valid for asphalt stabilized soils. If the influence of interfacial tensions between the various phases on the viscosity of the asphalt films is of such a magnitude so as to affect the effective stress parameters, than an analysis of experimental results should indicate the validity or non validity of the equations and the applicability of the method of determining values of X . In this investigation it will be assumed that the effective stress equations as applied to soils are also valid for asphalt stabilized soils and that the method

of determining values of the factor X for soils is also applicable to asphalt stabilized soils.

C. The Triaxial Compression Test

3.11 Triaxial Testing of Asphalt Stabilized Soils

The triaxial compression test is a mechanical test in which a load is applied to a cylindrical specimen while a supporting pressure is maintained against its sides by water, air or other means. The stress resistant properties of the material tested triaxially are derived from the relation between the testing load and the supporting pressure.

For approximately 30 years the attention of paving design engineers has been directed in part towards the use of triaxial shear strength methods to evaluate the engineering properties of asphalt mixtures. Two states in the United States, Kansas and Texas, base their design of asphalt mixtures on triaxial test results.

Two common empirical methods for the determination of the strength of asphalt mixtures are the Marshall stability test and the Hveem stabilometer method. Because of specimen dimensions and test procedures, the data from these tests cannot be subjected realistically to a theoretical analysis of stresses for purposes of design or research. In contrast the triaxial test is considered to be a rational approach to the investigation of asphalt mixtures and more closely similates field conditions (McLeod 1950). Gregg (1965) points out that the triaxial test yields information on some of the basic properties of an asphalt stabilized soil such as cohesion, internal friction and deformation characteristics.

He feels that it is the lack of this information which is responsible partly for the numerous test methods and design criteria in existence today and for the fact that asphalt stabilization is still very much an art.

If triaxial tests are performed on asphalt stabilized samples at two or more different confining pressures, the results may be presented in the form of the Mohr rupture envelope which is a graphical representation of the Coulomb equation

$$s = c + \tan \phi \qquad (7A)$$

used to define the shear strength of the particular samples. The test temperature, strain rate, degree of compaction and moisture content may be varied and closely controlled in triaxial testing. The triaxial test also allows for drainage controls and thus is the ideal test to be used in an investigation of the effective stresses of an asphalt stabilized soil.

Among the disadvantages inherent in the triaxial test are the cost and complexity of equipment, the number of specimens required and the fact that the method of testing is so flexible that at present there is no accepted standard procedure in the triaxial testing of asphalt mixtures. However, in comparison to other test methods for asphalt mixtures, the triaxial test evaluates strength properties under specified conditions and makes use of a strength theory that agrees closely to experimental results.

3.12 Some Aspects of Triaxial Test Procedures and Apparatus

Triaxial compression tests may be carried out under drained conditions, partly drained conditions or under undrained conditions. The main tests of interest in this investigation are the latter two. Partly drained tests (or consolidated undrained tests) are carried out on compacted samples which have been saturated and consolidated allowing drainage to occur. Axial loads are then applied and the sample is failed under undrained conditions with pore pressure measurements. Undrained tests are carried out on partly saturated samples and no drainage is allowed during the application of the confining pressure. Pore pressures due to volume decrease are built up during this process. The axial load is then applied and the sample is failed under undrained conditions with measurement of pore air and pore water pressures. The detailed procedures of the undrained and partly drained triaxial tests will not be given in this report. The procedures as presented by Bishop and Henkel (1962) are those which were generally followed in this investigation. Some other aspects which must be considered in triaxial testing include:

(a) Failure criteria in triaxial testing

Seed et al (1960) point out that it is often difficult, in examining the results of a triaxial test on a compacted sample, to assign a specific value to the soil strength or to decide at what point on the stress-strain curve failure has occurred. A commonly used measure of strength is the maximum deviator stress $(\sigma_1 - \sigma_3)_{max}$ which the sample can sustain. An alternate definition of strength is the maximum effective

stress ratio $\frac{\sigma_1'}{\sigma_3'}$ which represents the point of maximum obliquity of the resultant force on the failure plane and signifies that friction is fully mobilized. From triaxial test results on a fine sand, Bjerrum et al (1961) found that, for dense and medium dense sands, the maximum deviator stress and the principal effective stress ratio do not coincide. The maximum deviator stress continues to increase after the peak value of the principal stress ratio has been reached due to the dilatant structure of the sand.

(b) Experimental determination of factor X

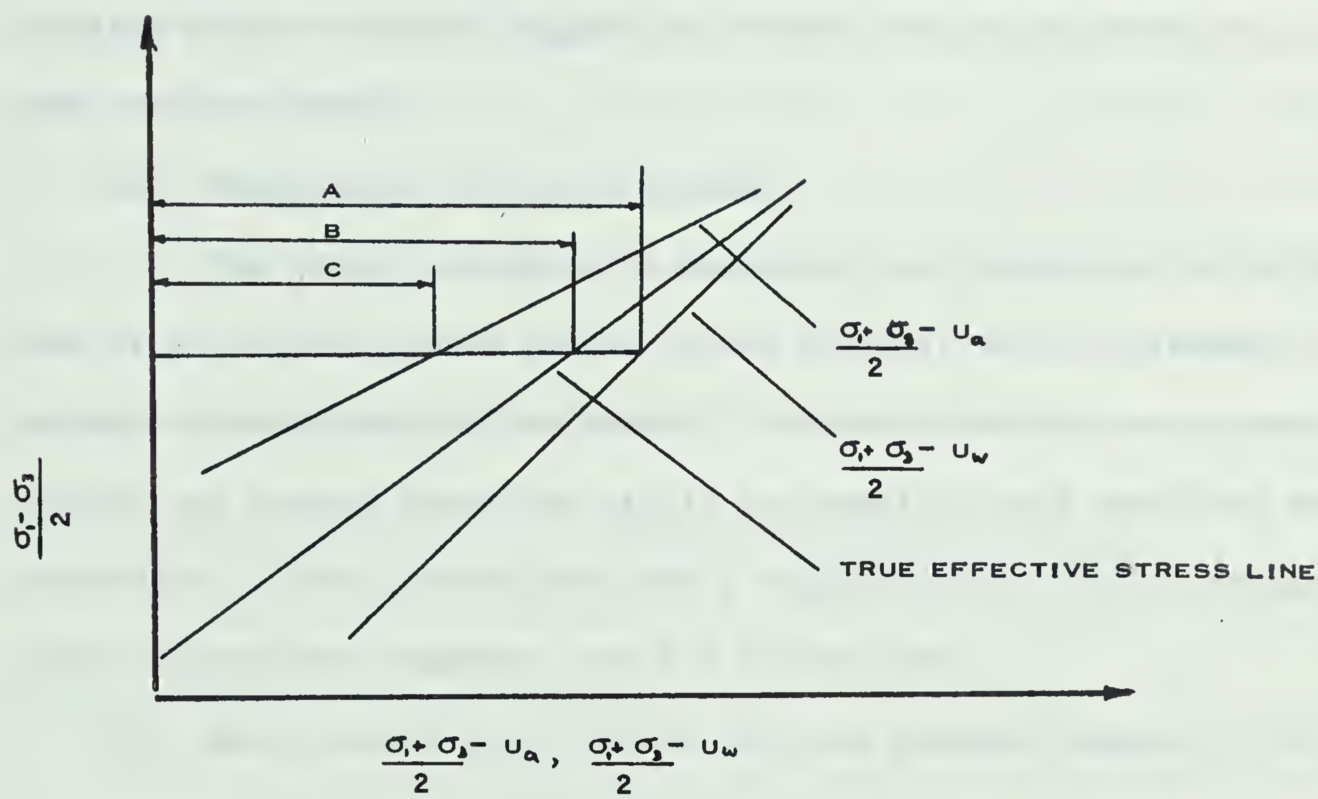
By using the results of triaxial tests on saturated samples and partly saturated samples, X may be determined by graphical means as shown in FIGURE 3. The Massachusetts Institute of Technology (1963) has derived an algebraic expression for computing X from the test results presented as shown in FIGURE 3. The resulting equation is

$$X = \frac{(\sigma_1 - \sigma_3)\ (\cot \beta - 1) - 2\ (\sigma_3 - u_a)}{2(u_a - u_w)}$$

where β = angle of modified Mohr envelope.

(c) Saturation of compacted samples

Many procedures are available for saturating a compacted sample, however whether full saturation is achieved is questionable. Lowe et al (1960) discuss the available methods of saturation and conclude that the most effective method is by using back pressures in conjunction with equal



$$X = \frac{B-C}{A-C}$$

FIGURE 3 - EXPERIMENTAL DETERMINATION OF FACTOR X [AFTER BISHOP ET AL 1960]

confining pressures. Tests were made on a compacted silty sand using back pressures of 35 psi and 100 psi, as well as the conventional methods of vacuum and percolation. The results showed that full saturation resulted only from the back pressure method and that back pressures in the range of 100 psi were required to saturate the samples. It was not determined however whether significant volume changes occurred during the back pressure method.

(d) Measurement of pore pressures

The common procedure of measuring pore pressures in the triaxial test is to utilize porous plates in the triaxial cell to transmit the pore pressure from the end of the sample. The end pressures are assumed to reflect the average pressures within the sample. Pore pressures must be measured by a device which requires a negligible pore volume change. The types of apparatus commonly used are (Sower 1963):

(1) Null systems that balance the pore pressure against a controlled measured pressure.

(2) Electrical transducers which require a negligible volume change and are sensitive to rapid changes in pressure.

The null system requires considerable attention and may allow the flow of pore fluid from or into the sample which usually results in a modification of the actual magnitude of the pore pressure readings. The electrical transducer on the other hand measures pressures allowing negligible flow and requires very little attention. The transducer is an electrical-mechanical device consisting of four unbonded strain gauges

connected in the form of a Wheatstone bridge so that any change in length, and consequently in resistance, in these gauges alters the electrical balance of the bridge. This produces a change in electrical signal in the output circuit which can be calibrated against applied pressures.

Pore water pressures are measured through a saturated fine porous plate which due to its high moisture retention capacity remains saturated with its moisture in pressure equilibrium with the pore water in the soil. Pore air pressures are measured through a porous element with a moisture retention capacity so low that it is unable to extract any moisture from the soil and remains dry. The air contained in the coarse element remains in pressure equilibrium with the pore air in the soil.

3.13 Limitations and Sources of Error in the Triaxial Test

Both from the engineering and from the scientific point of view the triaxial test has certain limitations and numerous sources of error. The limitations include a limited range of states of stress which can be studied and the fact that the intermediate principal stress is assumed equal to the minor principal stress. It is also assumed that apart from certain end effects a homogeneous state of stress is produced in the sample. The end restraints caused by the rigid end caps restrict lateral deformation and leads to a departure from the conditions of uniform stress at the ends of the sample. Bishop and Green (1965) conclude that no significant error occurs in the strength measurement of a sample provided the ratio of length to diameter is about 2 to 1. However, regardless of this ratio, end restraints cause non uniformity of volume strain, axial strain and pore

pressures, particularly at large strains. The limitations of the triaxial test are inherent in the test itself, and with the exception of end restraint, are not considered an important factor in this investigation.

The sources of error in a triaxial test are many and the principal ones that must be considered in this investigation are listed here:

(1) Membrane effect

The rubber membrane surrounding the sample is pushed into the crevices between the sample particles as the difference between the applied pressure and the pore pressures increase. This membrane effect results in the recording of erroneous volume changes for samples in a saturated and partly saturated condition. The membrane effect also aids in the compression of air in the partly saturated sample producing a pore air pressure which is not a direct result of the sample behavior. Haythornthwaite (1960) points out that the membrane effect is more important in coarse grained samples such as sands than in fine grained samples such as clays.

(2) Diffusion of air through the membrane

Diffusion of air through the rubber membrane surrounding a partly saturated sample causes the air pressure to drop. Blight (1960) observed that for tests lasting 100 minutes there was about one per cent drop in pore air pressure and 15 per cent drop for tests lasting one day. Poulos (1964) states that the changing of the membrane thickness does not appreciably alter the rate of air leakage. Air diffusion through the membrane can be reduced by using a liquid in which the solubility of air is practically negligible, such as mercury.

(3) Measurement of initial negative pressures in partly saturated samples.

The gradual development of negative pore-water and pore-air pressures with time in a sealed compacted sample was recognized by Bishop (1960). Hence before a triaxial test is carried out on a partly saturated sealed sample, the pore pressures must be allowed to come to equilibrium. Otherwise incorrect pore pressure readings will be recorded.

(4) Water leakage in triaxial testing can occur through the rubber membrane, through the bindings of the membrane to the cap and pedestal and through the packings of fittings and valves of the triaxial apparatus. Water leakage usually results in incorrect strength and pore pressure values.

(5) Cavitation of, and presence of air in, water of pore water pressure line.

When the negative pressure of the water in the pore water pressure line approaches zero (absolute) pressure, the water cavitates and succeeding pore water pressures are in error. Cavitation may also occur in water, in which air is present, at lower negative pressures and hence it is necessary to ensure air bubbles are absent from the pore water pressure line. The presence of air also affects the positive pore water pressure readings in that air is a compressible fluid and must be dissolved in the water before accurate pore pressure readings can be obtained.

CHAPTER IV

CLASSIFICATION AND ANALYSIS OF MATERIALS

4.1 Classification and Analysis of Materials

The soil used in this investigation was a fine, poorly graded sand from the Braeburn Pit (LSD 13-18-75-4-6) near Sexsmith, Alberta and was a sampled portion from the sand used by the Alberta Department of Highways for a soil cement stabilized base project (Project 2J-2). A microscopic examination of the soil showed the sand to consist mainly of spherical and subangular particles of quartz. TABLE II summarizes the classification and sieve analysis data of the sand. Actual sieve analysis, hydrometer, specific gravity, liquid limit and plastic limit results are given in Appendix A.

TABLE II

CLASSIFICATION AND ANALYSIS OF BRAEBURN FINES

Grain Size Distribution

U.S. Standard Sieve Size	Per cent Passing
4	100.0
20	99.8
40	91.2
60	43.4
100	19.5
200	9.9
Clay sizes (less than .002mm)	1.5

TABLE II (continued)

Sand Properties

AASHO Classification	A-3
Unified Classification	SP or SM
Specific gravity	2.67
Liquid limit W_L	26%
Plastic limit W_p	26.5%
Plasticity Index I_p	0
Uniformity coefficient	4

The asphalt cement used in this investigation was a sampled portion from an Alberta Department of Highways project (Project No. 2-D-3/2) and was supplied by the Husky Oil and Refining Company Limited of Lloydminster, Alberta. The viscosity of the asphalt at $140^{o}F$ (as determined by the Cannon-Manning Viscometer) was 510 poise. The penetration at $77^{o}F$ was 227 and thus was classified as a 200-300 Penetration Grade Asphalt Cement.

The samples were mixed and compacted with distilled water. Deaired water was used in the triaxial cell and in all water lines connected to the triaxial cell. Distilled water was used in the samples to guard against any reaction or exchange between minerals in the water and the asphalt-sand mix. Deaired water in the triaxial cell and the water lines was used to decrease the possibility of the existence of air bubbles in the triaxial test system.

CHAPTER V

TESTING PROGRAM AND PROCEDURES

5.1 General

This chapter summarizes the testing procedures and testing program followed in this investigation. Methods of sample preparation, compaction of samples and saturation of the samples are briefly described. A description of the triaxial test apparatus and associated equipment is also given. The testing program and primary objectives of this investigation are outlined in Section 5.7.

5.2 Preparation of Asphalt Stabilized Sand Samples

Samples were prepared using two different methods - the wet hot mix method and the foamed asphalt method. Two methods were used so as to provide a comparison for triaxial test results and hence an evaluation of the two different methods of mixing with regard to strength characteristics. Although the wet hot mix method is not considered to be of practical use in the field due mainly to economical considerations, it is useful as a research tool in that it provides samples which have stability characteristics similar to emulsified and cutback asphalt-sand mixes (Haas 1963). The amount of asphalt used in the asphalt-sand mixes was arbitrarily set at 4.2% of the dry weight of the sand and the water content chosen was approximately 14% of the dry weight of the sand. These quantities do not

represent the optimum conditions for the mix but these contents were considered sufficient for the purposes of this investigation.

The wet hot mix samples were prepared by heating the sand and the asphalt separately to 275°F, the suggested temperature for achieving an effective mix between the components. The required amount of asphalt was then added to the sand and mixed manually for 2 minutes. The mixture was allowed to cool at room temperature for 24 hours. The mixture of sand grains coated with asphalt was thoroughly broken up, mixed with 14% of water and was transferred to the moisture room for compaction.

The foamed asphalt samples were prepared by mixing the sand with the required amount of water and then placing the sand-water mix in the University of Alberta laboratory pugmill. In the meantime, foamed asphalt had been prepared at a temperature of 300°F. The pugmill was placed in operation and the foamed asphalt was distributed into the pugmill, through the foamed asphalt nozzle, in the form of an asphalt "mist". Mixing of the sand and foamed asphalt was continued for 2 minutes. The asphalt-sand mix was then allowed to cool in the moisture room for 24 hours after which compaction of the foamed asphalt samples was carried out.

5.3 Compaction of Samples

The method of compaction chosen in this investigation was that of kneading compaction using a 1.42 in. diameter mould and the compaction equipment associated with the miniature Harvard compaction apparatus. This method was chosen because of the immediate availability of equipment and the fact that the pedestal of the triaxial cell to be used was

1.42 in. Also, according to Schaub and Goetz (1961), kneading compaction seems to offer the best possibility for providing a desirable specimen "which is consonant with the present trend of thought regarding the formation of a specimen most nearly like the pavement prototype." Thomsen, Ferguson and Maze (1959) concluded that the miniature Harvard apparatus, using a 20 lb. spring compactor, closely approximates the Standard Proctor dry density of a sand. The diameter of the mould used in this investigation was only slightly larger than that of the miniature Harvard mould (1.42 in. vs 1.35 in.) and hence the dry densities obtained from this method of compaction may be assumed to approximate those of the Standard Proctor method. The maximum dry density obtained for the sand used in this investigation was 107.4 lbs/ft^3 at an optimum moisture content of 18%.

Since it was desirable to attain approximately the same dry densities for all samples, compactive efforts were varied to achieve this purpose. The following compactive efforts were utilized in the compaction of samples prepared by the various methods employed:

(1) Sand samples: 3 layers, 25 tamps per layer

(2) Foamed asphalt stabilized sand samples: 3 layers, 15 tamps per layer

(3) Wet hot mix stabilized sand samples: 5 layers, 50 tamps per layer

The final sample dimensions were 1.42 in. diameter and 2.84 in. high. The density gradients of compacted sand samples were investigated by determining the dry unit weights of one half inch strips along the length

of two samples. The dry unit weights were found to be constant within one per cent of the average dry unit weight. The density gradients of the asphalt stabilized sand samples were not investigated due to a shortage of samples.

5.4 Curing and Soaking of Asphalt Stabilized Sand Samples

A curing period of 7 days in a 100°F oven was chosen for this investigation mainly because past investigations at the University of Alberta have used this criterion for curing asphalt stabilized soils (Haas 1963, Laplante 1963). Jones (1962) showed that cured strengths in terms of total stresses levelled off after approximately 5 days. It is to be emphasized that this curing criterion is considered extreme and would not easily be obtained in practice. Also the fact that curing takes place from all faces of the sample means that field conditions are not simulated in this aspect of the testing. The cured strengths however represent the extreme upper limits of the strength values to be obtained from cured asphalt stabilized sand samples.

A soaking period of 7 days for cured samples was chosen arbitrarily to determine the changes in stability and volume incurred in the samples as a result of water adsorption. Cured samples were immersed in a tank of distilled water, the depth of water being maintained at 1/2 inch above the top of the specimens. Measurements and weights were taken before and after soaking. This soaking criterion is considered extreme and does not simulate actual field conditions. It does however represent the lower limits of strength values to be obtained from asphalt stabilized sand samples.

5.5 Triaxial Test Apparatus and Associated Equipment

The layout of the triaxial test apparatus and associated equipment used in this investigation is shown diagrammatically in FIGURE 4. A detailed sketch of the triaxial cell setup is shown in FIGURE 5. Briefly, the apparatus and equipment consisted of the following:

(a) Two constant pressure mercury control systems. The maximum pressure obtainable from each system was 110 psi.

(b) A twin burette volume change indicator manufactured by Wykeham Farrance Engineering Ltd., Slough, England. This apparatus was capable of measuring unlimited volume changes because of the fact that volume change readings could be reversed.

(c) Triaxial cell No. 156, 150 psi capacity, manufactured by Leonard Farnell and Co. Ltd., Hatfield, England.

(d) Loading machine of 2000 lb. capacity and 25 different rates of strain, manufactured by Wykeham Farrance Engineering Ltd.

(e) Proving ring No. 1516, 1000 lb. capacity, manufactured by Wykeham Farrance Engineering Ltd.; proving ring No. 4789, 500 lb. capacity, manufactured by Soiltest Inc., Chicago.

(f) Two electrical transducers manufactured by Dynisco, Cambridge, Massachusetts. No. 28514, capacity 0-30 psia; No. 22792, capacity 0-100 psia.

(g) Switch and balance unit, model SB-1, manufactured by Budd Instruments Division, Phoenixville, Pennsylvania.

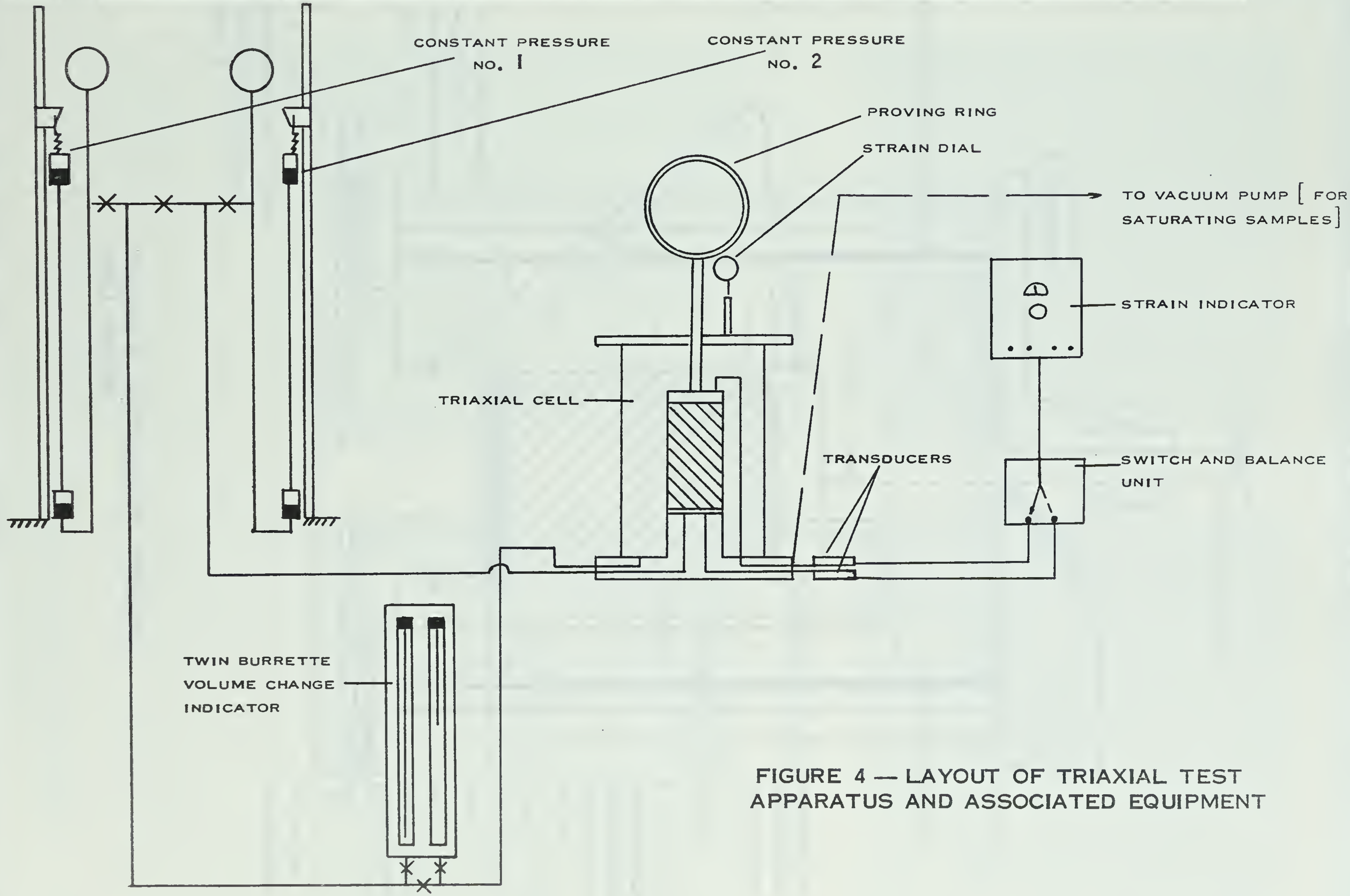


FIGURE 4 — LAYOUT OF TRIAXIAL TEST APPARATUS AND ASSOCIATED EQUIPMENT

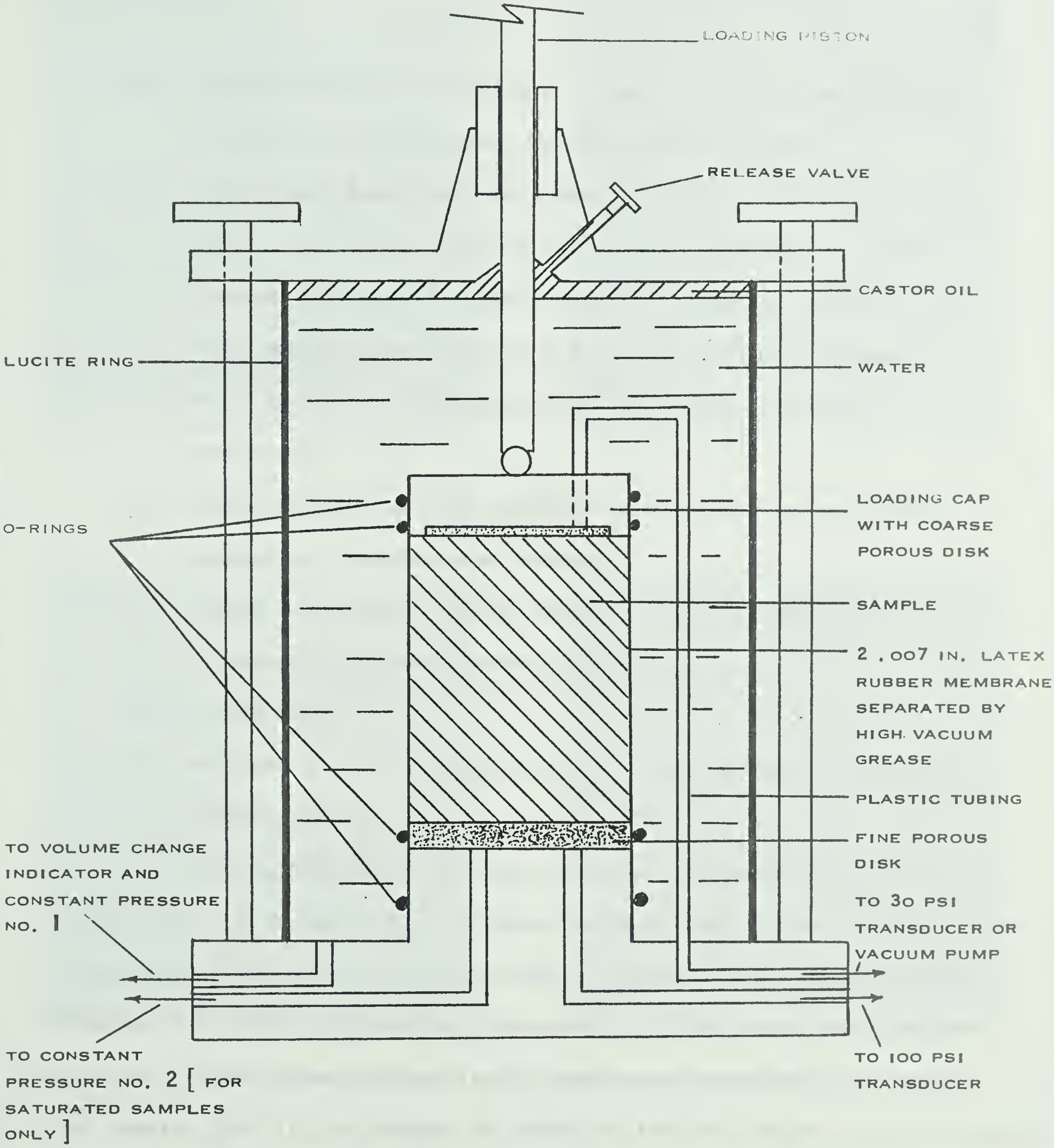


FIGURE 5 - DETAILS OF TRIAXIAL
CELL SETUP

(h) Baldwin SR-4 strain indicator, Type No. 562644, manufactured by Baldwin-Lima-Hamilton, Waltham, Massachusetts.

(j) Fine porous base disk, Celloton CF 26 with a porosity of 46%, an air entry value of 14.1 psi, a permeability coefficient of 1×10^{-6} cm/sec, 1.42 in. diameter, .39 in. thick and a maximum pore size of 2.9 microns. (This information was supplied by the manufacturer Aerox Ltd., Tamworth, Scotland).

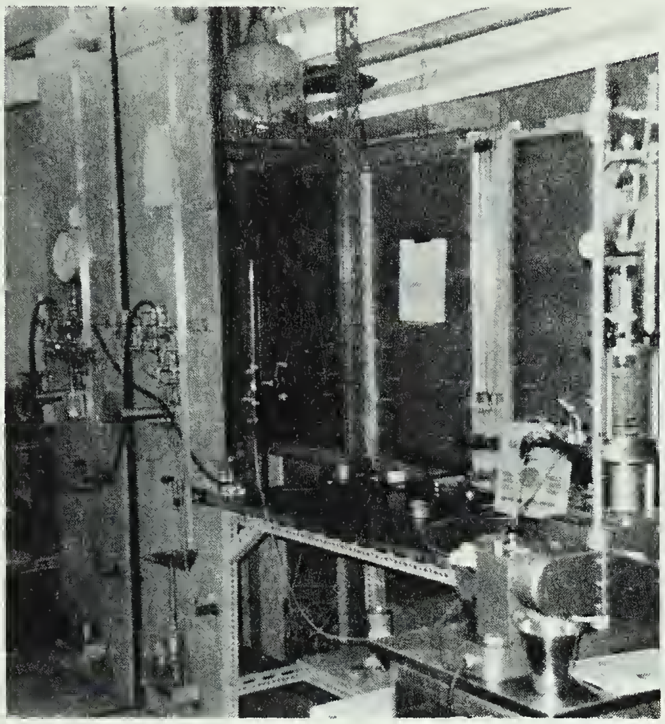
(k) Coarse porous top disk, negligible air entry value, highly permeable. Manufacturer unknown.

(l) Rubber membranes, .007 in. thick, 1.4 in. diameter, manufactured by Wykeham Farrance Engineering Ltd.

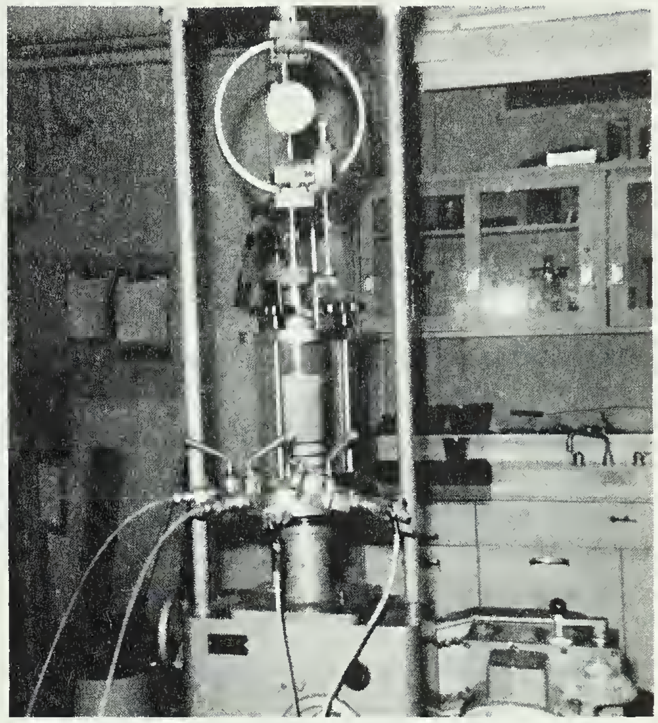
(m) Vacuum pump

(n) O-rings, 1.25 in. inside diameter, loading cap and flexible plastic tubing.

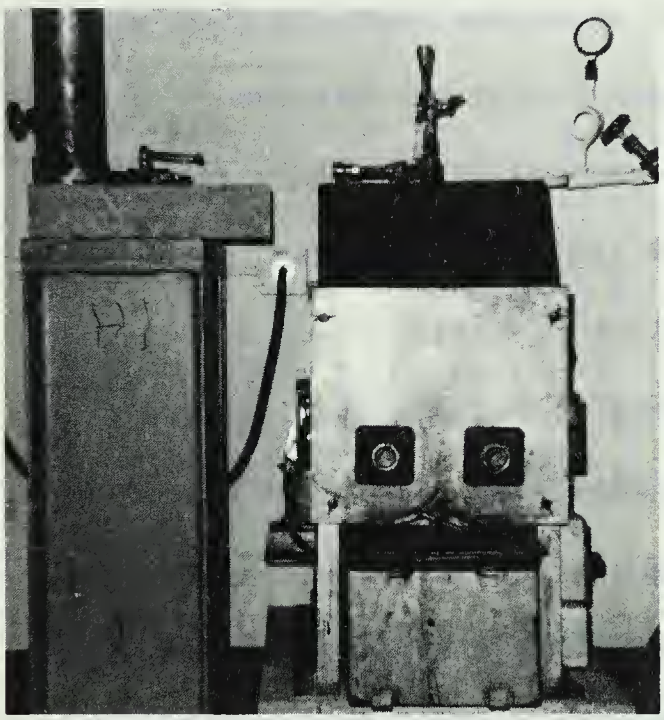
Calibration charts for the electrical transducers are given in Appendix B at a reduced scale to those actually used in the testing program. Calibration charts used for the proving rings were those provided by the Department of Civil Engineering, University of Alberta and were accepted as true. Since volume changes in the sample were measured from outside the sample, that is, by changes in volume of the cell water, it was necessary to calibrate the change in volume of the lucite ring of the triaxial cell and the associated tubing with different applied cell pressures. This calibration is given in Appendix B.



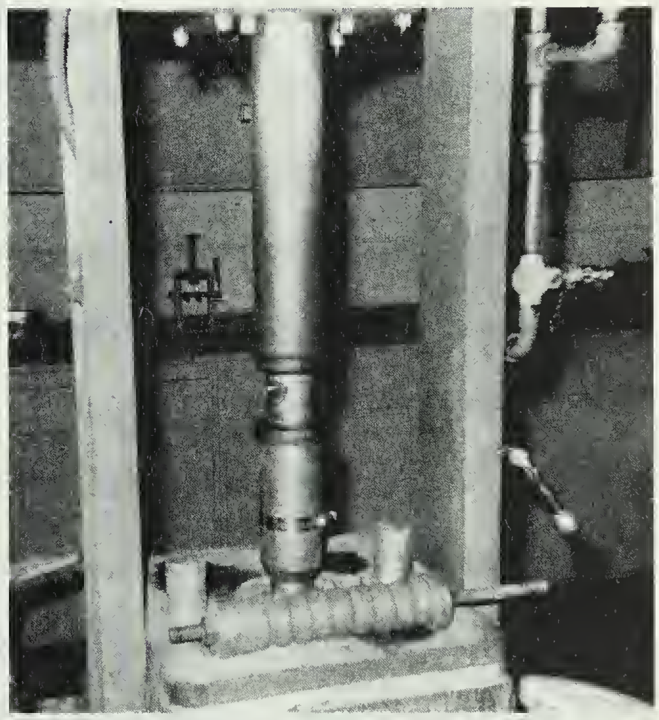
(A) GENERAL VIEW OF TRIAXIAL TEST SET UP



(B) TRIAXIAL CELL AND LOADING MACHINE



(C) U OF A PUGMILL WITH FOAMED ASPHALT ATTACHMENT



(D) COMPACTION APPARATUS

PLATE 1 - TRIAXIAL TEST APPARATUS AND EQUIPMENT USED IN TESTING PROGRAM

5.6 Triaxial Test Procedures

(a) Saturation of compacted samples:

Difficulty was encountered in developing a suitable method of saturating the compacted samples. Many methods were tried, including percolation of water from the base upwards through the sample, and the vacuum-percolation method. Using these methods of saturation the pore pressure response, from an incremental cell pressure increase of 20 psi under undrained conditions, ranged from 40 to 50%, which denotes a low degree of saturation. The method finally decided upon was a modification of the back pressure method as outlined by Lowe and Johnson (1960).

Briefly the method consisted of setting up the compacted sample in the triaxial cell in the usual manner. With all connections to the cell closed, a vacuum of 4 psi was applied to the sample through the top porous disk for a period of 15 to 20 minutes. Water under a pressure of 4 psi was then allowed to percolate through the base plate up through the sample and at the same time the cell pressure was increased to 4 psi. The cell pressure and back pressure were then increased simultaneously at 4 psi increments with sufficient time allowed for pressures to come to equilibrium throughout the sample. This procedure was continued to a predetermined pressure of 70 psi. It could not be determined whether volume changes occurred during saturation and this is considered a main drawback in the procedure. The time required for this procedure was approximately 12 hours.

(b) Consolidated-undrained tests on saturated samples.

After saturation of the compacted samples was completed, consolidated-undrained tests were carried out on the saturated samples. Due to the limitation of the pressure control system, only consolidation pressures up to 40 psi could be utilized. Consolidation was carried out against the saturation back pressure of 70 psi and was considered to be complete when the pore pressure reading under undrained conditions was zero, the zero point coinciding with the 70 psi backpressure. The time for consolidation was approximately one hour. The sample saturation was then tested by increasing the cell pressure by 20 psi under undrained conditions. If a pore pressure response of 90% or over was obtained in one minute, the sample was assumed to be fully saturated. The sample was then sheared to failure at a strain rate of .002 inches per minute or approximately 7% per hour, pore water pressure readings and volume changes being recorded at appropriate strain intervals.

(c) Undrained tests on partly saturated samples

In these tests the compacted samples were set up in the triaxial cell in the usual manner and the initial pore water pressure and pore air pressure were permitted to come to equilibrium over a period of approximately 20 hours. Undrained tests were then performed utilizing cell pressures of up to 40 psi, pore water and pore air pressure readings being recorded as the test proceeded. The strain rate used was the same as that for saturated samples.

5.7 Outline of Testing Program and Objectives

The testing program consisted of triaxial compression tests on compacted sand samples and compacted asphalt stabilized sand samples. It was decided to use only three to four tests for each series to define the Mohr rupture envelope due to the time required for each test and the limited time available for testing. The results from the triaxial tests were presented as points on a modified Mohr envelope thus avoiding the confusion of superimposed Mohr circles. The common Mohr rupture envelope was obtained by applying Equations 8A and 8B (refer to Section 3.5) to the results obtained from the modified Mohr envelope.

The objectives of the testing program were as follows:

(1) To determine the shear strength, in terms of effective stresses, of the asphalt stabilized sand samples in a saturated state and to compare the strengths obtained for the two methods of asphalt stabilization and those obtained for compacted sand samples.

(2) To devise a testing procedure, utilizing available equipment at the University of Alberta, for determining the shear strength, in terms of effective stresses, of the partly saturated compacted sand and asphalt stabilized sand samples.

(3) To determine the effects of curing and soaking on the shear strength, in terms of effective stresses, of the compacted asphalt stabilized sand samples in a saturated and partly saturated condition.

(4) To investigate the volume changes and pore pressures which occur in the compacted sand and asphalt stabilized sand samples during shearing.

The testing program followed in this investigation is summarized in TABLE III. The program is divided into two main series - triaxial tests on saturated samples (S series) and tests on partly saturated samples (U series).

TABLE III

SUMMARY OF TESTING PROGRAM

Test Series	Sample No.	Description
SA	1 to 4	Compacted sand samples; tested in saturated condition. Cell pressure at 10 psi increments from 10 to 40 psi.
SB	5 to 7	Compacted foamed asphalt stabilized sand samples; tested in saturated condition. Cell pressure at 10 psi increments from 10 to 30 psi.
SC	8 to 10	Compacted wet hot mix stabilized sand samples; tested in saturated condition. Cell pressure at 10 psi increments from 10 to 30 psi.
SD	11 to 13	Compacted foamed asphalt stabilized sand samples, cured; tested in saturated condition. Cell pressure at 10 psi increments from 10 to 30 psi.
SE	14 to 16	Compacted wet hot mix stabilized sand samples, cured; tested in saturated condition. Cell pressure at 10 psi increments from 10 to 30 psi.
SF	17 to 19	Compacted foamed asphalt stabilized sand samples, cured, soaked; tested in saturated condition. Cell pressure at 10 psi increments from 10 to 30 psi.
SG	20 to 22	Compacted wet hot mix stabilized sand samples, cured, soaked; tested in saturated condition. Cell pressure at 10 psi increments, from 10 to 30 psi.

TABLE III (continued)

Test Series	Sample No.	Description
UA	23 to 25	Compacted sand samples; tested in partly saturated condition. Cell pressure at 10 psi increments from 20 to 40 psi.
UB	26 to 29	Compacted foamed asphalt stabilized samples, cured; tested in partly saturated condition. Cell pressure at 10 psi increments from 20 to 40 psi. Sample No. 26 not sheared.
UC	30 to 33	Compacted wet hot mix asphalt stabilized sand samples, cured; tested in partly saturated condition. Cell pressure at 10 psi increments from 20 to 40 psi. Sample No. 30 not sheared.
UD	34 to 36	Compacted foamed asphalt stabilized sand samples, cured, soaked; tested in partly saturated condition. Cell pressure at 10 psi increments from 10 to 30 psi.

CHAPTER VI

RESULTS AND DISCUSSION OF RESULTS

A. Presentation of Results

6.1 Summary of Results

TABLE IV shows a summary of the test results. The cohesion and friction angle values listed were derived from the modified Mohr envelopes obtained for each test series by applying Equations 8A and 8B (refer to Section 3.5) to the results obtained from the modified Mohr envelopes. Dry unit weights listed in this table are the total dry unit weights based on the dry unit weight of the compacted sand-asphalt combination. Degrees of saturation are expressed as the volume of the voids of the compacted samples and were calculated using the sample dimensions, moisture contents and the respective specific gravities of the sand and asphalt. The percent swell is the volume expansion expressed as a percentage of the volume of the cured specimen. Sample calculations are given in Appendix C.

The results obtained from the testing program are presented in graphical form in FIGURES 6 to 15. FIGURES 6 to 10 show the results obtained for the saturated (S) series and FIGURES 11 to 14 show the results obtained for the partly saturated (U) series. FIGURE 15 summarizes the results of the testing program in the form of Mohr rupture envelopes.

TABLE IV

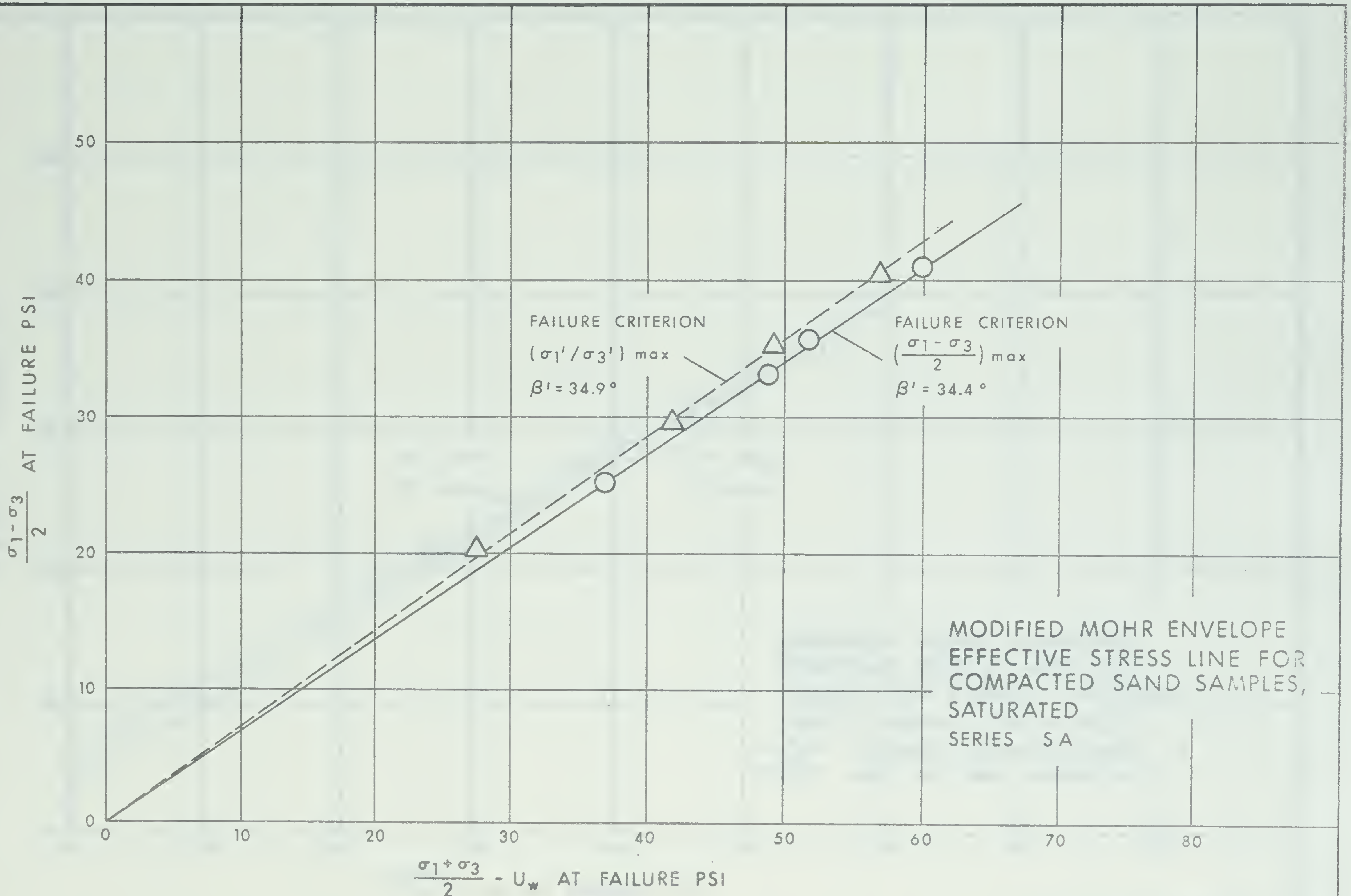
SUMMARY OF TEST RESULTS

Saturated Series

Test Series	d' psi	c' psi	β' Deg.	ϕ' Deg.	Av. Total Dry Unit Wt. Prior To Shear Test lbs/ft^3	Av. Calculated Deg. of Sat. After Testing %	Av. Pore Pressure Response %	Av. % Swell After Soaking	Remarks
SA	0	0	34.4	43.2	101.6	106.8	92.5		Compacted sand series
SB	6	7.2	29.0	33.7	103.0	111.1	98.0		Compacted foamed asphalt series
SC	4	5.0	30.6	36.3	102.1	113.2	92.1		Compacted wet hot mix series
SD	6	7.2	29.4	34.2	103.3	117.4	94.0		Cured foamed asphalt series
SE	4	5.0	30.7	36.4	102.1	120.3	95.0		Cured wet hot mix series
SF	6	7.2	29.6	34.5	98.8	104.8	93.8	4.77	Soaked foamed asphalt series
SG	4	5	30.2	35.7	97.8	112.2	95.5	4.04	Soaked wet hot mix series

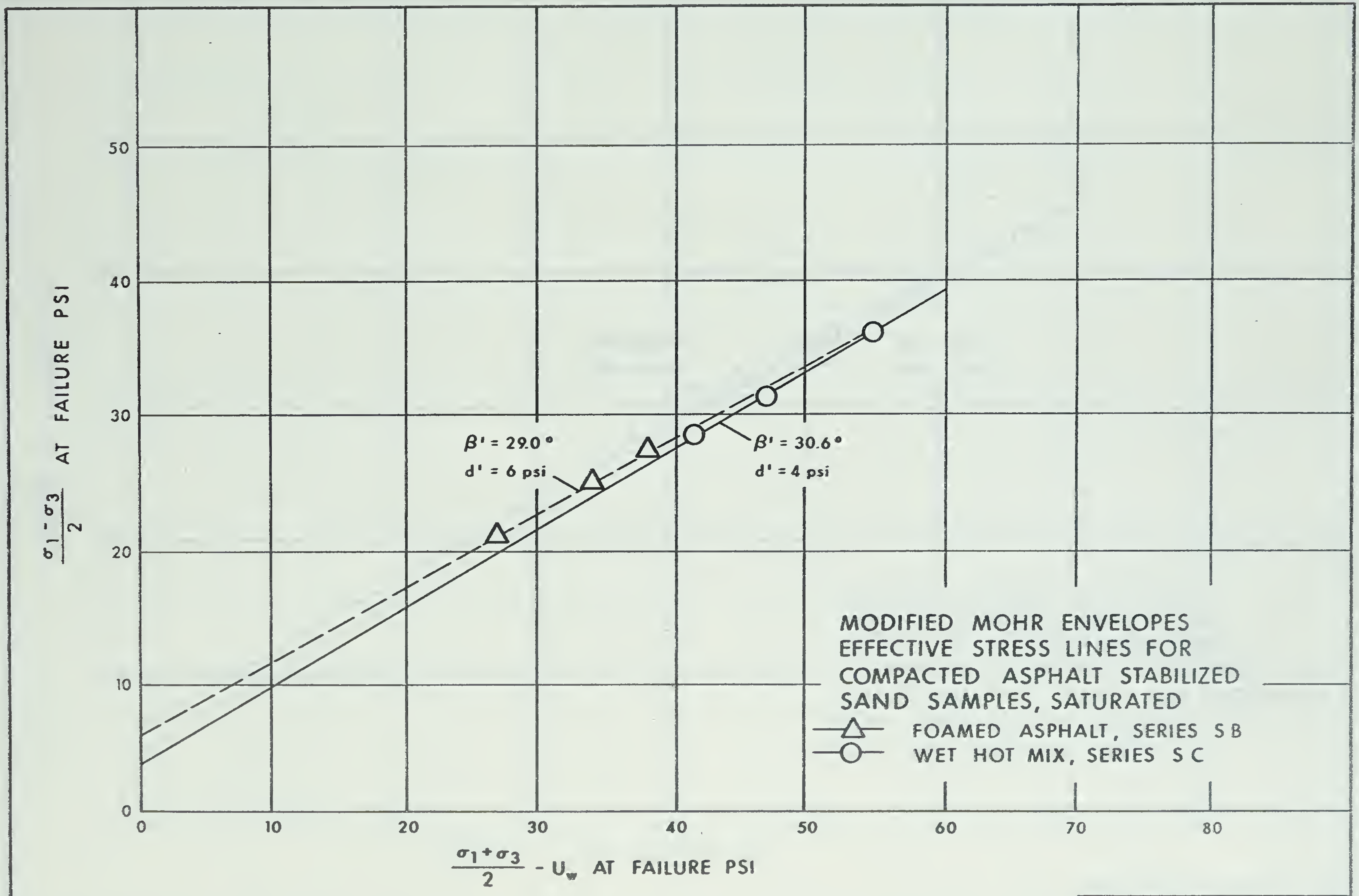
Partly Saturated Series

Test Series	d psi	c psi	β Deg.	ϕ Deg.	Av. Total Dry Unit Wt. Prior To Shear Test lbs/ft^3	Av. Calculated Deg. of Sat. %	Av. % Swell After Soaking	Remarks
UA	3	3.5	30	35.4	102.30	68.9		Compacted sand series
UB	41	48.8	28.6	32.7	103.85	8.0		Cured foamed asphalt series
UC	28.3	34.3	29.4	34.3	102.16	7.1		Cured wet hot mix series
UD	7.0	8.0	25.6	28.6	99.96	65.89	4.73	Soaked foamed asphalt series



MODIFIED MOHR ENVELOPE
EFFECTIVE STRESS LINE FOR
COMPACTED SAND SAMPLES,
SATURATED
SERIES SA

FIGURE 6



$\frac{\sigma_1 - \sigma_3}{2}$ AT FAILURE PSI
$\frac{\sigma_1 + \sigma_3}{2} - U_w$ AT FAILURE PSI
0
10
20
30
40
50
60
70
80
$\beta' = 29.0°$
$d' = 6$ psi
$\beta' = 30.6°$
$d' = 4$ psi
MODIFIED MOHR ENVELOPES
EFFECTIVE STRESS LINES FOR
COMPACTED ASPHALT STABILIZED
SAND SAMPLES, SATURATED
FOAMED ASPHALT, SERIES S B
WET HOT MIX, SERIES S C

FIGURE 7

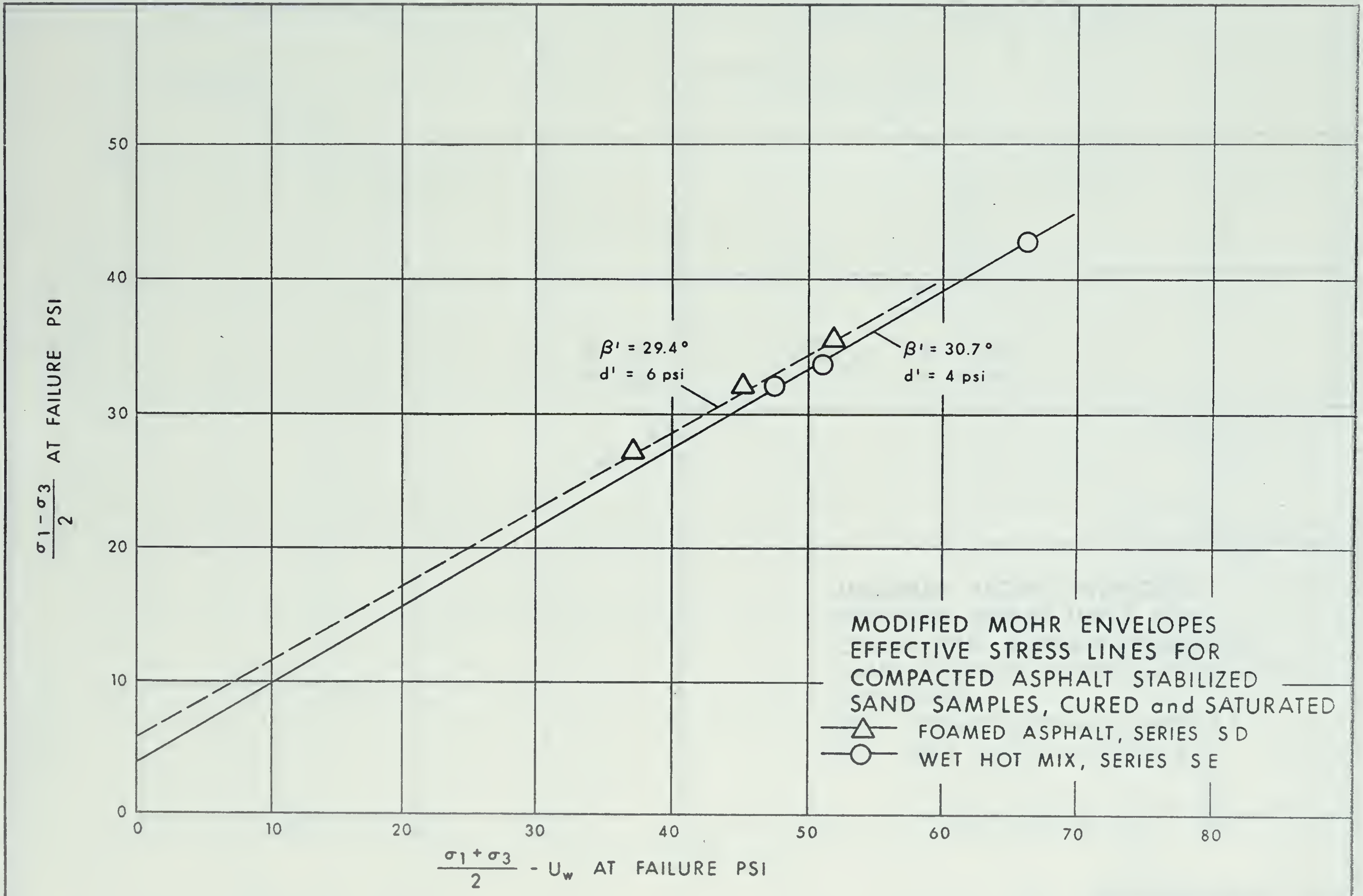

MODIFIED MOHR ENVELOPES
EFFECTIVE STRESS LINES FOR
COMPACTED ASPHALT STABILIZED
SAND SAMPLES, CURED and SATURATED
FOAMED ASPHALT, SERIES SD
WET HOT MIX, SERIES SE
$\beta' = 29.4°$
$d' = 6$ psi
$\beta' = 30.7°$
$d' = 4$ psi
$\frac{\sigma_1 - \sigma_3}{2}$ AT FAILURE PSI
$\frac{\sigma_1 + \sigma_3}{2} - U_w$ AT FAILURE PSI
0
10
20
30
40
50
60
70
80

FIGURE 8

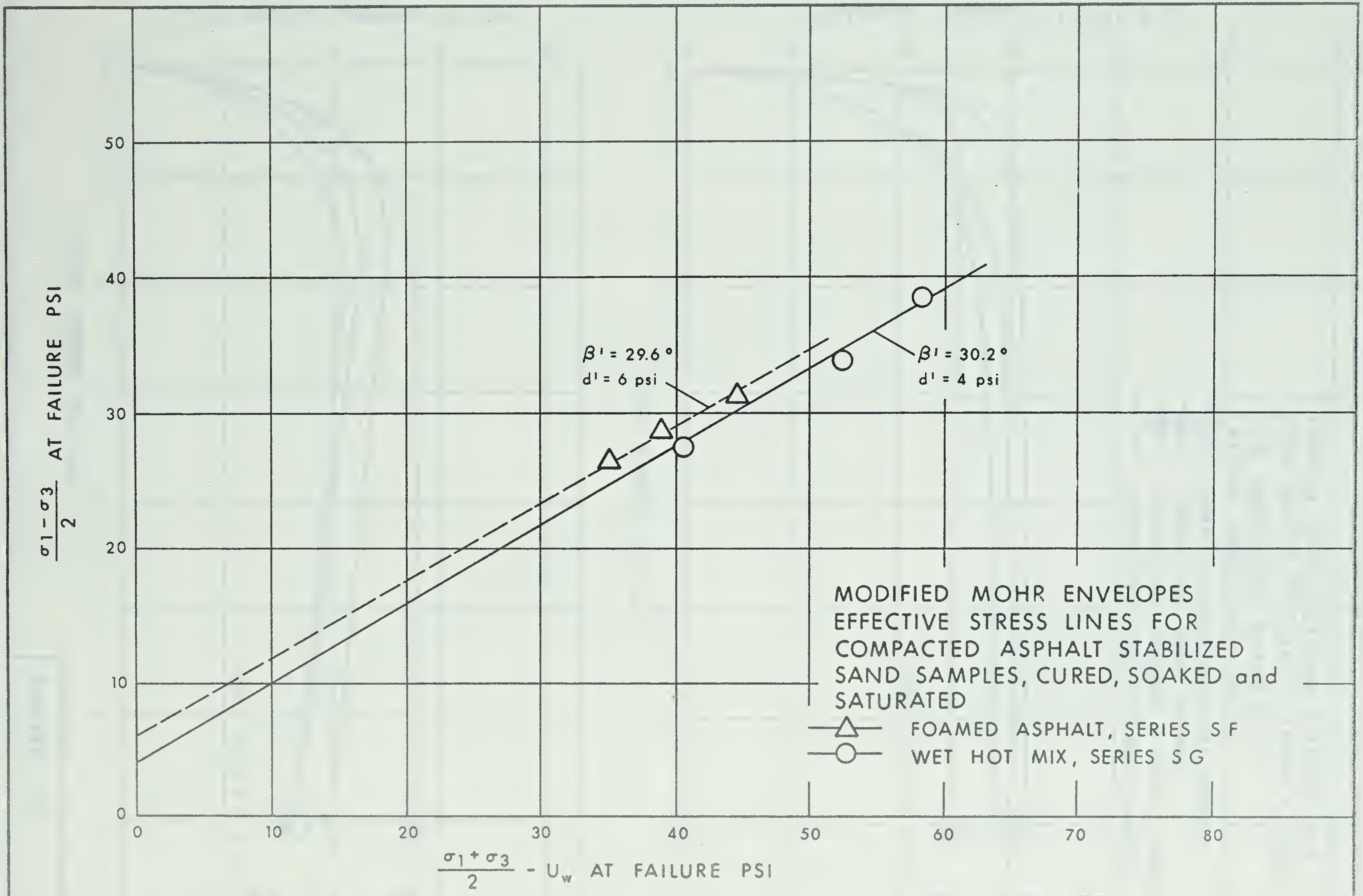

50
40
30
20
10
0
10
20
30
40
50
60
70
80
$\frac{\sigma_1 - \sigma_3}{2}$ AT FAILURE PSI
$\frac{\sigma_1 + \sigma_3}{2} - U_w$ AT FAILURE PSI
$\beta' = 29.6°$
$d' = 6$ psi
$\beta' = 30.2°$
$d' = 4$ psi
MODIFIED MOHR ENVELOPES
EFFECTIVE STRESS LINES FOR
COMPACTED ASPHALT STABILIZED
SAND SAMPLES, CURED, SOAKED and
SATURATED
FOAMED ASPHALT, SERIES SF
WET HOT MIX, SERIES SG

FIGURE 9

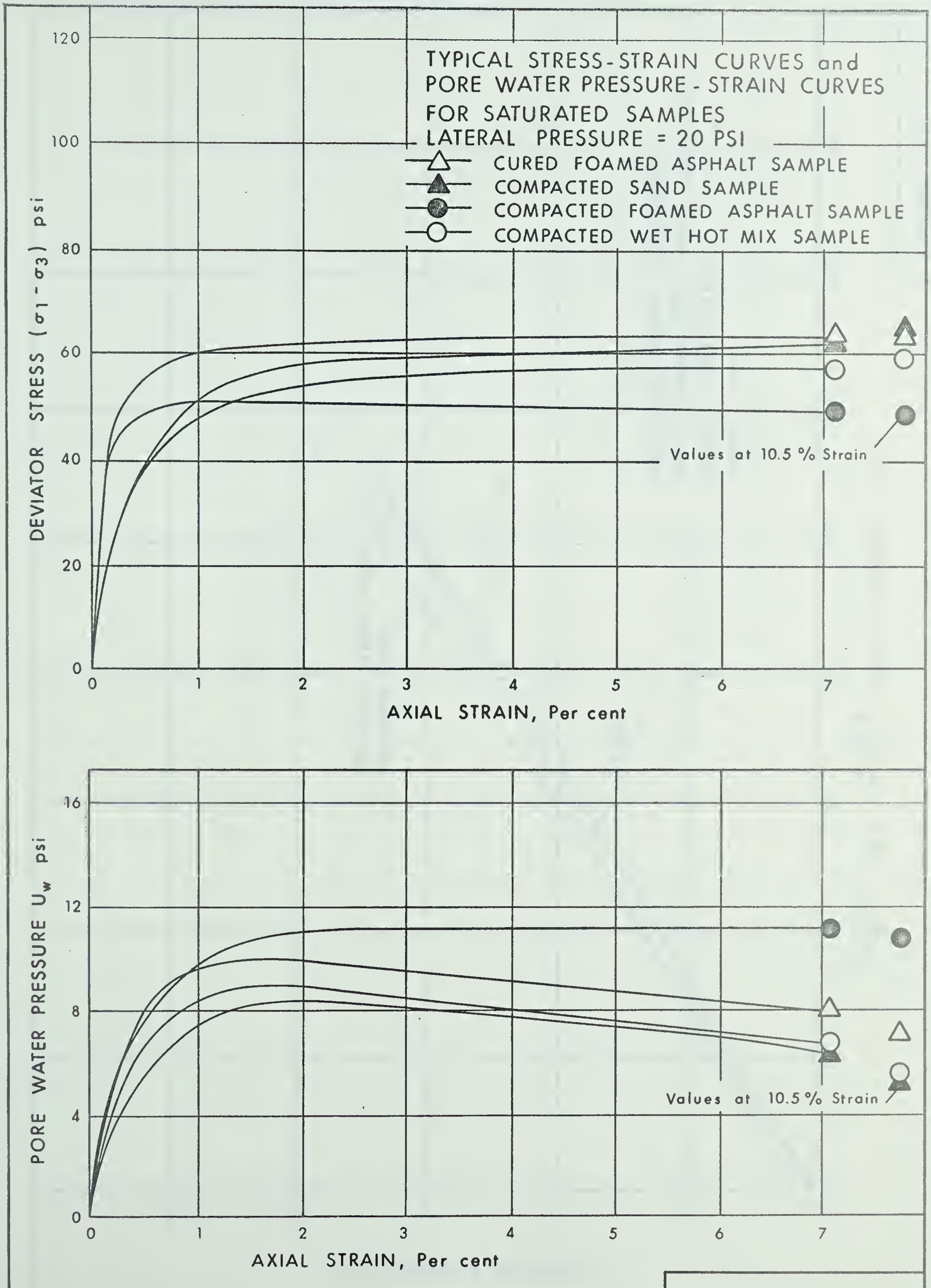
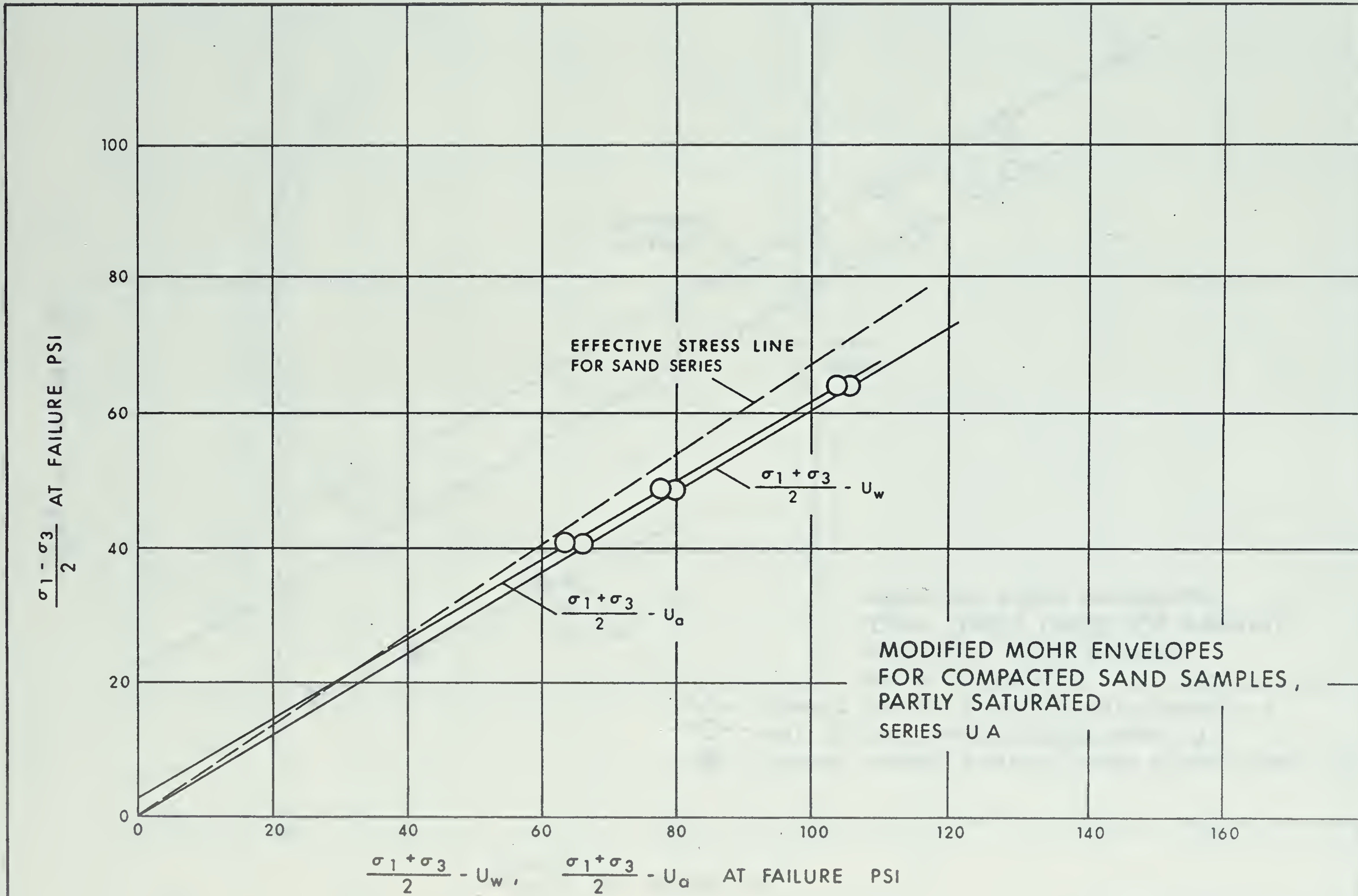

TYPICAL STRESS-STRAIN CURVES and
PORE WATER PRESSURE - STRAIN CURVES
FOR SATURATED SAMPLES
LATERAL PRESSURE = 20 PSI
CURED FOAMED ASPHALT SAMPLE
COMPACTED SAND SAMPLE
COMPACTED FOAMED ASPHALT SAMPLE
COMPACTED WET HOT MIX SAMPLE
DEVIATOR STRESS ($\sigma_1 - \sigma_3$) psi
120
100
80
60
40
20
0
1
2
3
4
5
6
7
Values at 10.5 % Strain
AXIAL STRAIN, Per cent
PORE WATER PRESSURE U_w psi
16
12
8
4
0
Values at 10.5 % Strain
AXIAL STRAIN, Per cent

FIGURE 10



MODIFIED MOHR ENVELOPES
FOR COMPACTED SAND SAMPLES,
PARTLY SATURATED
SERIES U A

FIGURE 11

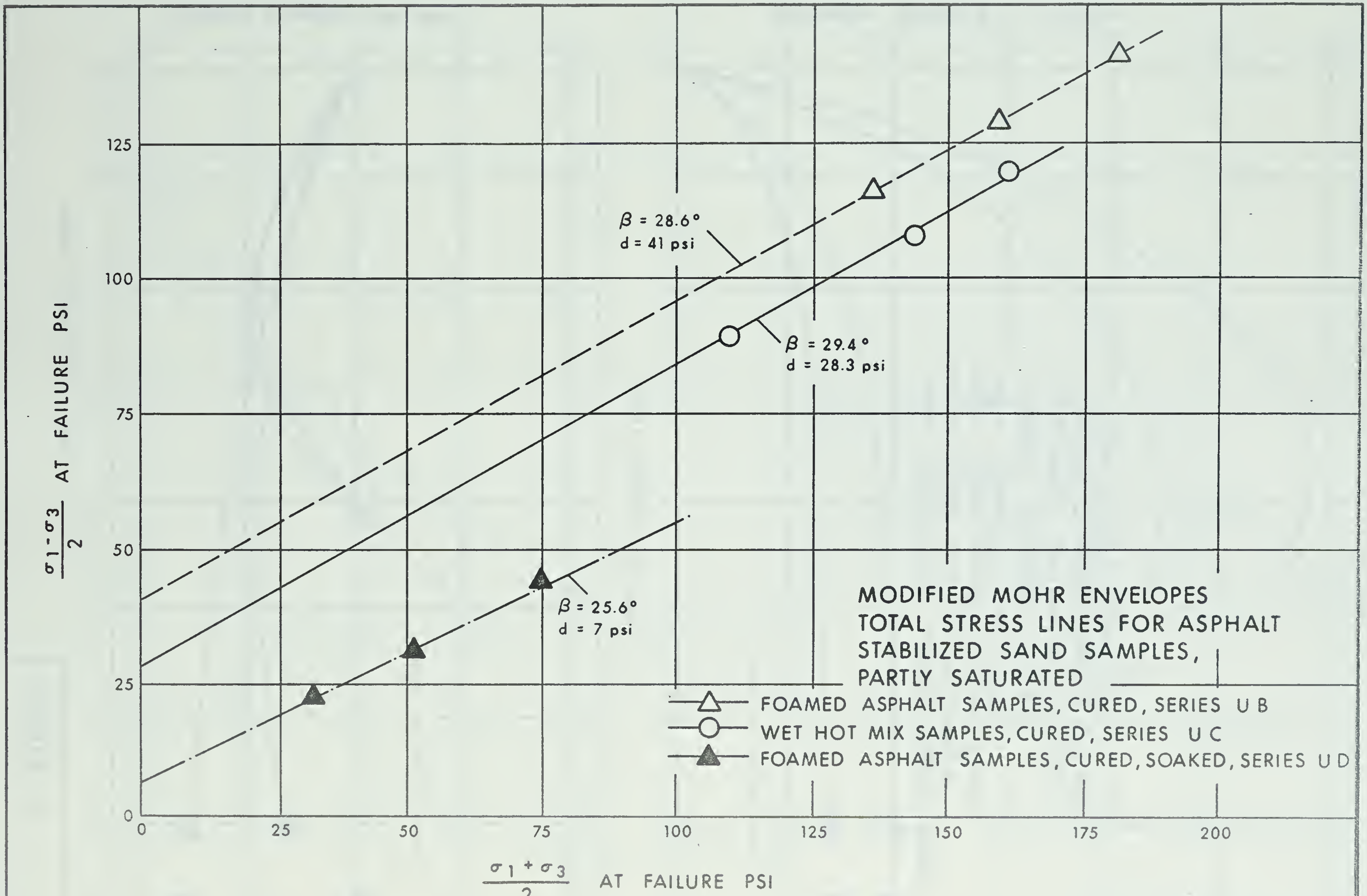
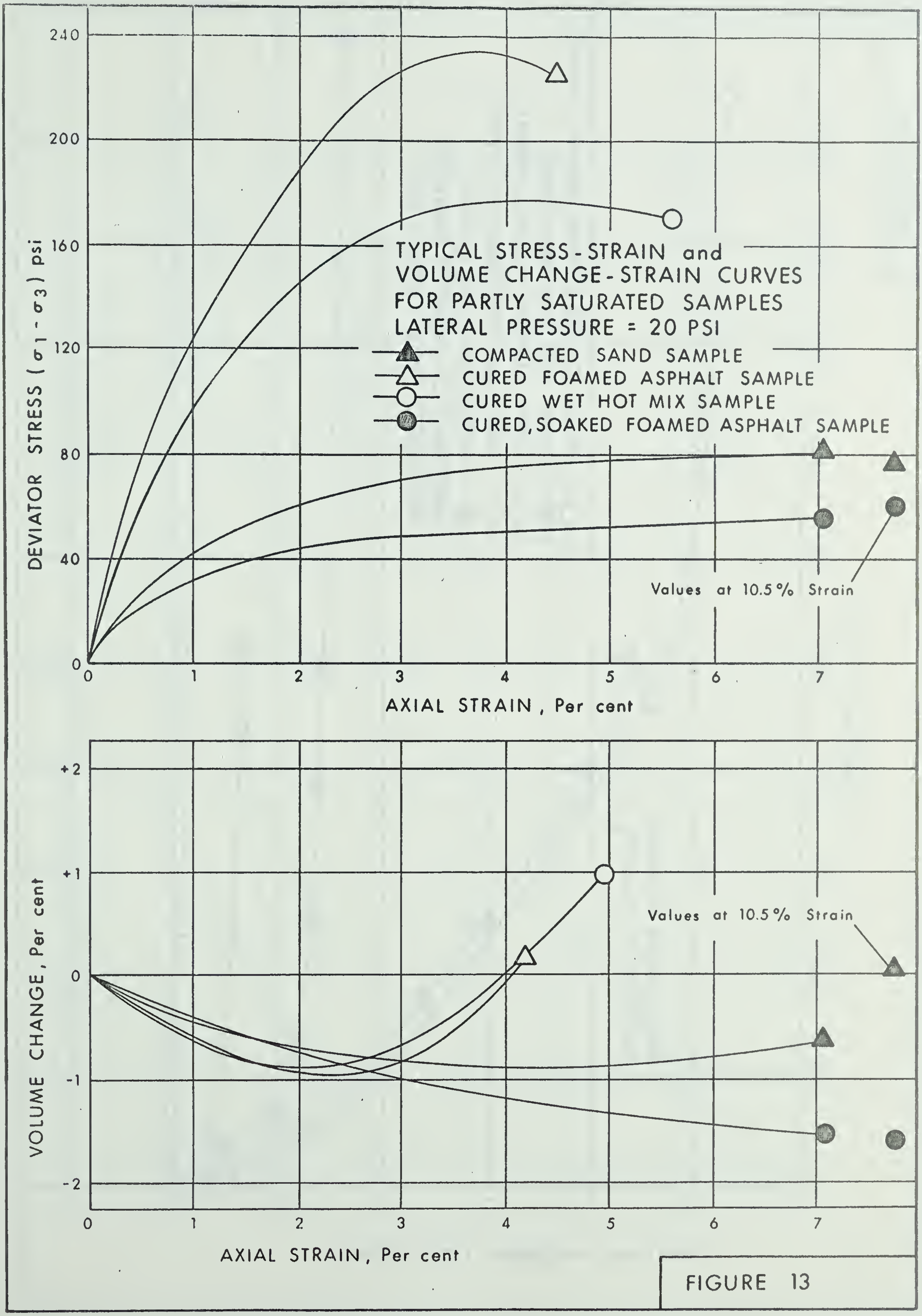

MODIFIED MOHR ENVELOPES
TOTAL STRESS LINES FOR ASPHALT
STABILIZED SAND SAMPLES,
PARTLY SATURATED
FOAMED ASPHALT SAMPLES, CURED, SERIES U B
WET HOT MIX SAMPLES, CURED, SERIES U C
FOAMED ASPHALT SAMPLES, CURED, SOAKED, SERIES U D
β = 28.6°
d = 41 psi
β = 29.4°
d = 28.3 psi
β = 25.6°
d = 7 psi
$\frac{\sigma_1 - \sigma_3}{2}$ AT FAILURE PSI
$\frac{\sigma_1 + \sigma_3}{2}$ AT FAILURE PSI
0
25
50
75
100
125
150
175
200

FIGURE 12



240
200
160
120
80
40
0
DEVIATOR STRESS $(\sigma_1 - \sigma_3)$ psi
TYPICAL STRESS - STRAIN and
VOLUME CHANGE - STRAIN CURVES
FOR PARTLY SATURATED SAMPLES
LATERAL PRESSURE = 20 PSI
COMPACTED SAND SAMPLE
CURED FOAMED ASPHALT SAMPLE
CURED WET HOT MIX SAMPLE
CURED, SOAKED FOAMED ASPHALT SAMPLE
Values at 10.5 % Strain
0 1 2 3 4 5 6 7
AXIAL STRAIN, Per cent
+2
+1
0
-1
-2
VOLUME CHANGE, Per cent
Values at 10.5 % Strain
0 1 2 3 4 5 6 7
AXIAL STRAIN, Per cent

FIGURE 13

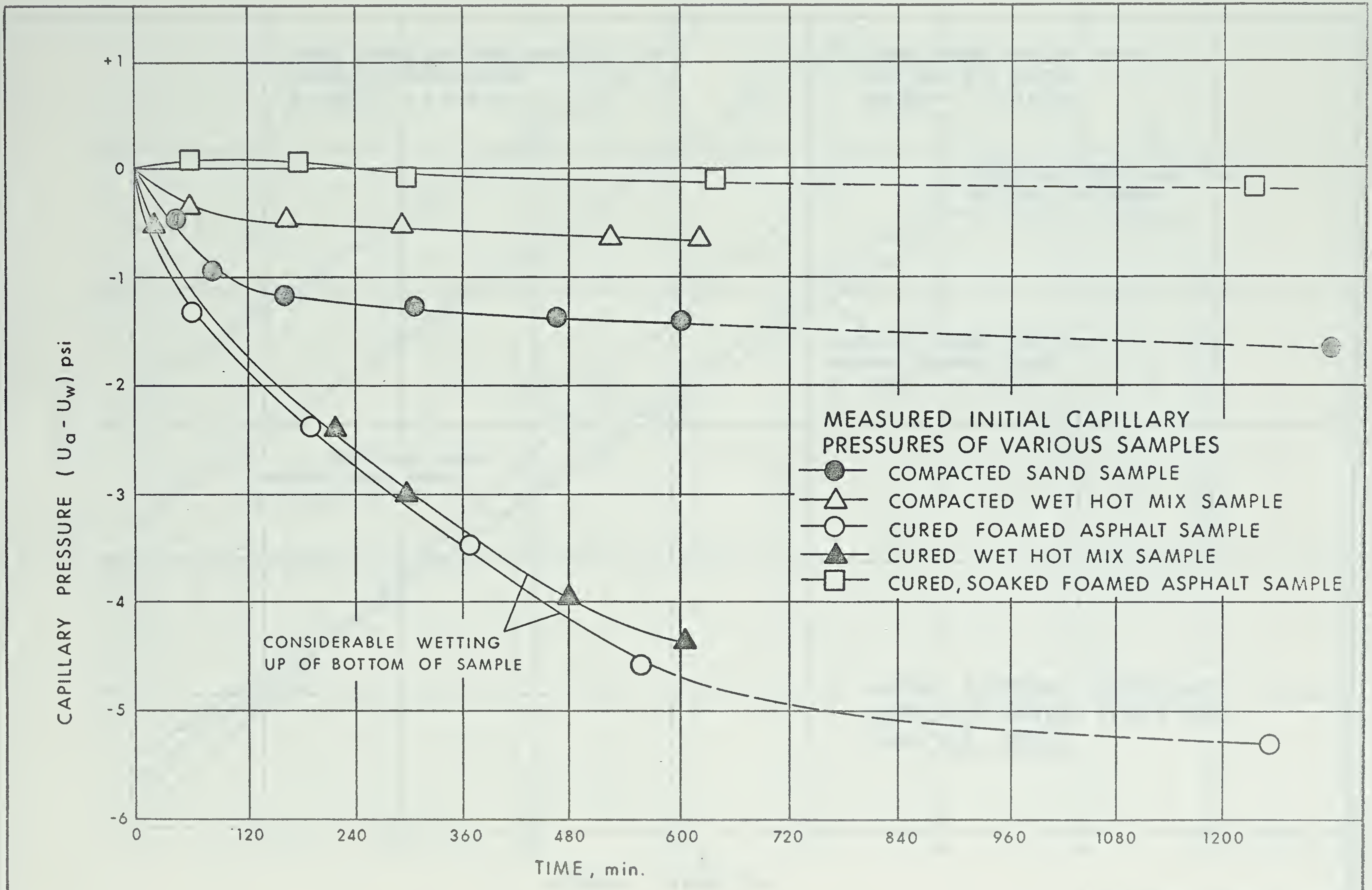

MEASURED INITIAL CAPILLARY PRESSURES OF VARIOUS SAMPLES
COMPACTED SAND SAMPLE
COMPACTED WET HOT MIX SAMPLE
CURED FOAMED ASPHALT SAMPLE
CURED WET HOT MIX SAMPLE
CURED, SOAKED FOAMED ASPHALT SAMPLE
CONSIDERABLE WETTING UP OF BOTTOM OF SAMPLE
CAPILLARY PRESSURE (Ua - Uw) psi
+1
0
-1
-2
-3
-4
-5
-6
0
120
240
360
480
600
720
840
960
1080
1200
TIME, min.

FIGURE 14

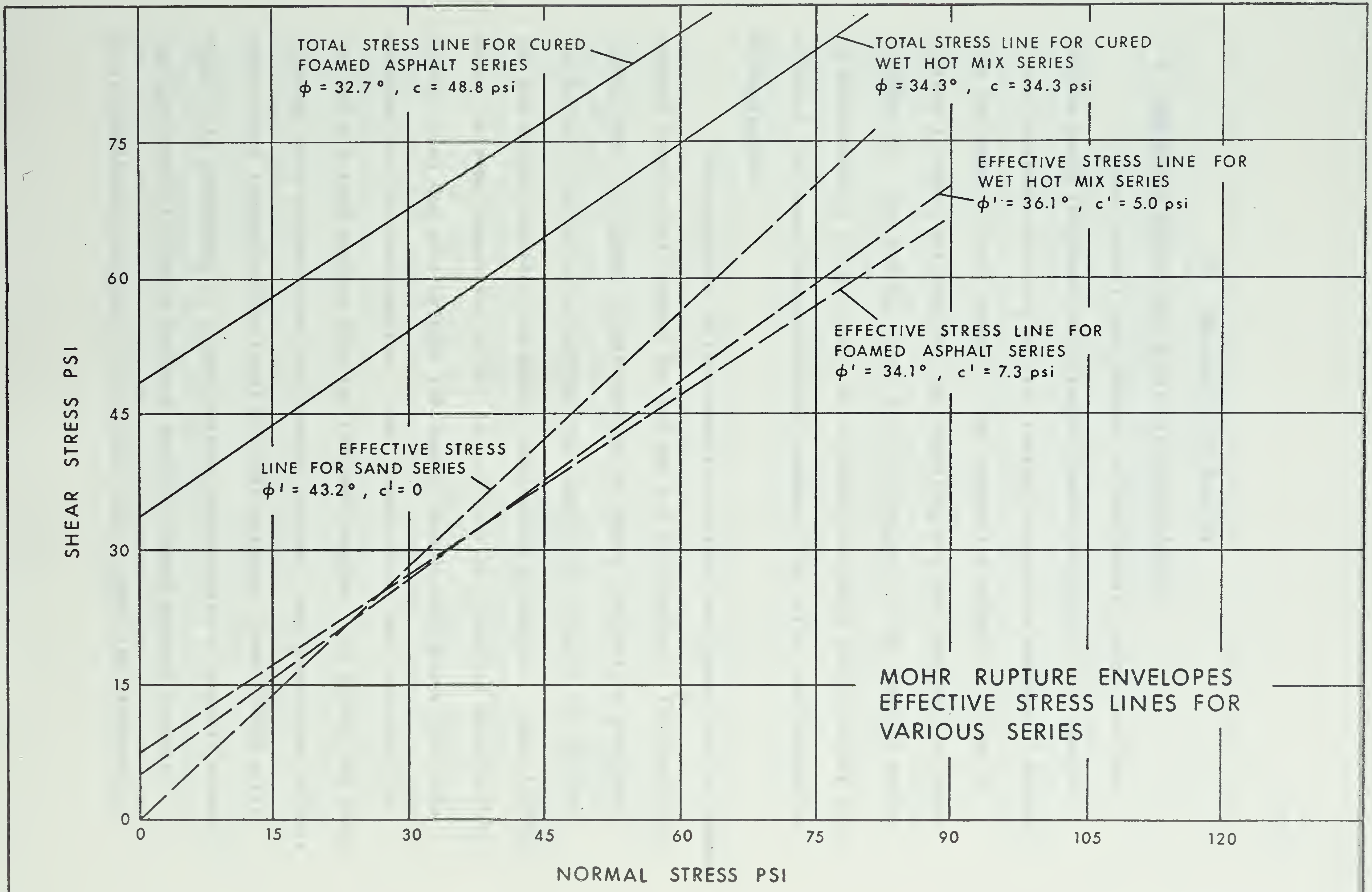
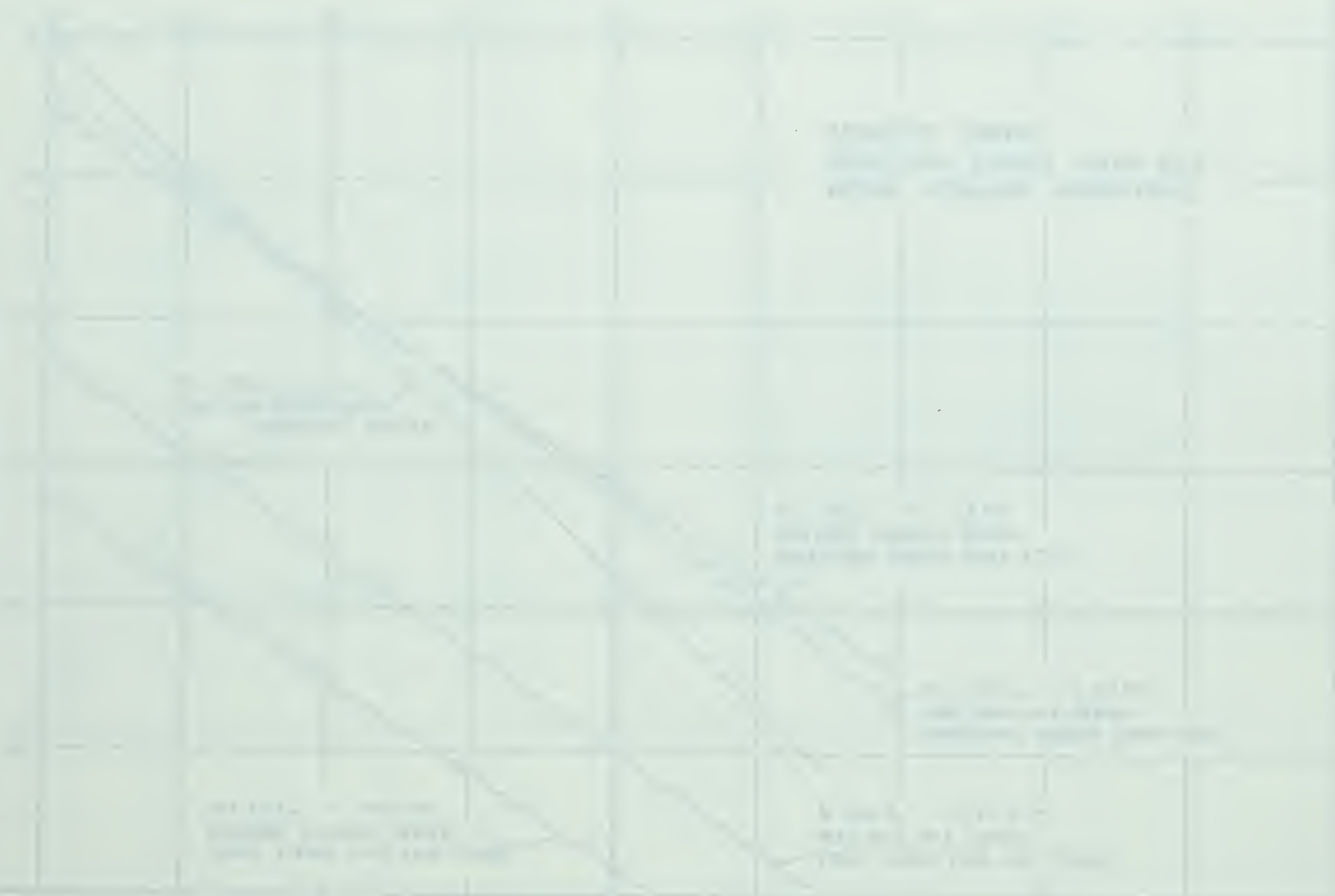

TOTAL STRESS LINE FOR CURED FOAMED ASPHALT SERIES
ϕ = 32.7°, c = 48.8 psi
TOTAL STRESS LINE FOR CURED WET HOT MIX SERIES
ϕ = 34.3°, c = 34.3 psi
EFFECTIVE STRESS LINE FOR WET HOT MIX SERIES
ϕ' = 36.1°, c' = 5.0 psi
EFFECTIVE STRESS LINE FOR FOAMED ASPHALT SERIES
ϕ' = 34.1°, c' = 7.3 psi
EFFECTIVE STRESS LINE FOR SAND SERIES
ϕ' = 43.2°, c' = 0
MOHR RUPTURE ENVELOPES
EFFECTIVE STRESS LINES FOR VARIOUS SERIES
SHEAR STRESS PSI
NORMAL STRESS PSI
0
15
30
45
60
75
90
105
120

FIGURE 15



B. Discussion of Results

6.2 Compactive Effort and Total Dry Unit Weights

Since the dry unit weights of the asphalt stabilized sand samples would influence the strength characteristics of the samples, it was desirable to carry out the testing program on samples of essentially the same dry unit weights. The compactive efforts outlined in Section 5.3 were utilized and the resulting dry unit weights are shown in TABLE IV. The average total dry unit weights of the samples varied by approximately 1.5 per cent and although the small variation may have affected the shear strength values somewhat, the effect was assumed to be negligible.

The magnitude of the compactive effort was lowest for the foamed asphalt samples and highest for the wet hot mix samples. This would indicate that the foamed asphalt method of sample preparation provided the better lubricating properties for compaction purposes and the wet hot mix method provided the poorer lubricating properties. Haas (1963), using the same compactive effort for all samples, obtained considerable higher total dry unit weights for foamed asphalt-sand mixes than were obtained for the sand samples. Also, lower values were obtained for the wet mix-sand samples than for the sand samples. In effect the same findings were evident from this testing program, only compactive efforts were varied instead of the total dry unit weights. Haas (1963) attributed this phenomenon to a more uniform distribution of asphalt around the sand particles which resulted from the foamed asphalt process. It is considered necessary to also take into account the viscosity of

the asphalt phase to explain this phenomenon. The foamed asphalt-sand mix, before compaction, was maintained at a constant moisture content in the moisture room. Although the asphalt phase of the mix was allowed to cool, water was not permitted to escape. The asphalt, at the time of compaction, appeared to be in the form of an emulsion thus allowing for a more efficient and easier packing of the particles. This would imply that the asphalt did not regain its original viscosity and that this process is dependent upon drying conditions and time. The wet hot mix method of sample preparation resulted in the asphalt phase having a higher viscosity than was evident in the foamed asphalt process. Also, the addition of water after the sand-asphalt mix was achieved resulted in what appeared to be repulsive forces between the asphalt and water phases as it was very difficult to mix the water with the sand-asphalt mixture. A higher compactive effort was required to attain approximately the same total dry weight as obtained for the foamed asphalt-sand mix. This is believed to have been due to the higher viscosity of the asphalt and to the repulsive forces between the asphalt and water phases.

It is not entirely clear why the sand samples required a higher compactive effort than the foamed asphalt-sand mix. This may be partly accounted for by the fact that the foamed asphalt mix consisted of a greater quantity of fluids or part fluids (water and asphalt) than did the sand-water mix, thus providing a greater degree of lubrication and a better orientation of the particles. Also the physical chemical effects of the air-water-sand interfaces existing in the sand mix as opposed to the effects of the air-water-sand-asphalt phases existing in the foamed

asphalt-sand mix may have influenced the better orientation of the particles.

6.3 Discussion of Results - Saturated Series

(a) Evaluation of Test Procedures and Equipment

Before the results from the tests on the saturated samples are discussed, it is considered necessary to evaluate the testing procedures used in obtaining these results. Referring to TABLE IV it is noted that the average calculated degrees of saturation after testing for the saturated test series are all in excess of 100 per cent. Since this is physically impossible, the testing procedure used is open to question. Degrees of saturation were based on the original dimensions of the samples prior to setting them up in the triaxial cell, taking into account the volume change due to consolidation. Based on the calculated degrees of saturation, it is apparent that the original volume of the samples increased during the testing of the samples. There was no evidence of volume changes taking place during the undrained portion of the triaxial test. Volume changes in the cell water were recorded during this phase of the testing and when the displacement of cell water due to piston action was taken into account the net volume change of the sample was negligible. If volume changes in the sample took place it is believed they occurred during the saturation phase of the testing. As outlined in Section 5.6, saturation was achieved using the back-pressure method. In this method, the pore water pressure and the cell water pressure were maintained at equilibrium but due to the limitations of the testing

equipment volume changes could not be detected and thus could not be controlled because of the lack of detection. During the process of saturation the capillary pressures in the compacted samples would be released allowing for volume expansion of the sample. Volume expansion was noted for cured samples which were allowed to soak (refer to TABLE IV, Test Series SF and SG) and ranged from 4 to 5 per cent. It is thus concluded that volume increases in the samples could very well have taken place during the saturation process. Whether the final results obtained from the triaxial tests on saturated samples were affected by volume increase in the samples could not be determined. A method of detecting volume changes during saturation of the compacted samples would have been desirable but no method could be devised during the testing program. In the discussion of triaxial test results on saturated samples it is assumed that the incurred volume changes due to the saturation process had negligible effects on the final results.

All other aspects of the test procedures used for the triaxial tests on the saturated series were common procedures as outlined in many references including Bishop and Henkel (1962). Although these procedures were mainly concerned with the testing of soils, it was assumed that they would be also applicable to asphalt stabilized soils.

(b) Effective Stress Lines from Modified Mohr Envelopes

FIGURE 6 shows the effective stress line for the compacted sand samples (Series SA). The two failure criteria are compared in this figure and a difference of $.5^{\circ}$ for β' values is evident in the results. This

difference is due to the dilatant characteristics of the compacted sand samples and is compatible with results obtained by Bjerrum et al (1961) from tests on fine sands. Whereas the difference between the two failure criteria is small and whereas Bishop and Henkel (1962) and Wissa and Ladd (1964) recommend that the maximum principal stress ratio not be used as a failure criterion for soils with a cohesion intercept, it was decided to base the failure criterion for all tests on the maximum deviator stress.

FIGURE 7 shows the effective stress lines obtained for compacted wet hot mix samples and compacted foamed asphalt samples (Series SB and SC). The presence of asphalt resulted in an appreciable reduction in the modified effective angle of internal friction β' and induced a small amount of effective cohesion d' into the samples. Similar findings were evident from tests performed by Fossberg (1964) using a cutback asphalt as the stabilizing binder for a fine sand. A difference in effective strength parameters for the two types of stabilized soil is also apparent and this is believed to be accounted for by the two methods used in the preparation of the samples. The foamed asphalt method of sample preparation was expected to give a better distribution of asphalt throughout the samples. This was due to the fact that the asphalt was introduced in the form of an asphalt "mist" and the resulting low viscosity and surface tension would allow a better distribution of asphalt over the sand particles. The wet hot mix method of sample preparation was expected to give the poorer distribution of asphalt due to the fact that the hot asphalt was introduced into the hot sand mass without the presence

of water. The wetting ability of asphalt is appreciably decreased with the absence of water (Benson and Becker 1942) and thus the distribution of asphalt over the sand particles was expected to be inferior to that obtained by the foamed asphalt method. The foamed asphalt samples were thus expected to display a higher d' value due to the anticipated better distribution of asphalt and the resulting improved bonding action. The results shown in FIGURE 7 confirm this prediction and also show a slightly lesser β' value for the foamed asphalt samples. However the small differences encountered in this particular test series would lead one to conclude that the method of preparation did not seriously affect the effective strength characteristics of the asphalt stabilized samples.

FIGURE 8 shows the results from triaxial tests on cured asphalt stabilized samples (Series SD and SE). The values of β' and d' are essentially the same as those obtained for the as-compacted samples of the previous series. It is evident from these results that curing did not induce permanent physical changes in the shear strength characteristics of the asphalt stabilized samples, in terms of effective stresses. FIGURE 9 shows that soaking of the cured asphalt stabilized samples (Series SF and SG) did not induce permanent physical changes in the strength properties of the samples as once again the modified effective stress parameters are essentially the same as the results obtained from tests on the cured and as-compacted samples. In the cured and soaked test series the small difference in the values of β' and d' of the two types of mixture remained basically constant.

According to Endersby (1942) an initial decrease of water content through curing of an asphalt stabilized soil results in permanent resisting characteristics which an uncured asphalt stabilized soil will not have. This view is also supported by such authors as Herrin (1958) and Abdel-Hady and Herrin (1965) but it is to be noted they are speaking in terms of total stresses and of samples that were partly saturated. There was no evidence from tests on saturated samples to support this view when speaking in terms of effective stresses.

The soaking tests on the asphalt stabilized samples showed that there was no evidence of stripping action by water on the asphalt films. It stripping had occurred there would probably have been an increase in β' and a decrease in d'. Stripping is caused by the interaction of forces among the sand particles, the water and the asphalt phases. Although the soaking period for the samples was not of long duration as compared to practical conditions, the severity of the soaking was believed to have been suitable for stripping action.

(c) Stress-Strain Curves

FIGURE 10 shows typical stress-strain curves for some of the saturated samples with a lateral confining pressure of 20 psi. Shown in conjunction with these curves are the corresponding pore water pressure curves. From the stress-strain curves it is noted that the deviator stress of the foamed asphalt samples approached their maximum deviator stress $(\sigma_1 - \sigma_3)_{max}$ at a strain of less than 1 per cent whereas the wet hot mix and sand samples approached $(\sigma_1 - \sigma_3)_{max}$ more gradually. This

is believed to have been due to the effect of a better distribution of asphalt and the better bonding of the particles in the foamed asphalt samples. The pore water pressure (u_w) curves give an indication of the volume changes that would have taken place in the samples if drainage was permitted. Increasing values of u_w denote a tendency for volume reduction while decreasing values of u_w indicate a tendency for volume expansion. Using this criterion, the sand sample was observed to have dilatant properties, that is, after a tendency for initial volume decrease the sample tended to increase in volume. The asphalt stabilized sand samples showed a tendency for greater initial volume decrease in that measured initial pore pressures were higher than those measured for the sand samples. The as-compacted foamed asphalt sample displayed the characteristics of a normally consolidated clay in that there was very little tendency for the sample to dilate. The pore water pressure increased to a maximum and this maximum value was maintained throughout the duration of the test. This was probably due to the lowered viscosity of the asphalt which provided a greater lubricating influence and a tendency for continuing volume decrease. The cured foamed asphalt sample showed dilatant properties comparable to the sand and wet hot mix samples although the tendency for initial volume decrease was greater than that for the sand and wet hot mix samples.

The curves shown in FIGURE 10 are typical for the samples tested in a saturated condition. The pore water pressure curves are not consistent with published results for sand and sand-asphalt mixes in a dense or medium dense state (Bishop and Henkel 1962, Bjerrum et al 1961,

Schaub and Goetz 1961). The sand samples in this testing program possessed an initial void ratio of .60 and the sand-asphalt samples an initial void ratio of .50. These initial void ratios denote fairly dense samples and hence negative pore water pressures were expected to be prevalent during shearing. The absence of negative pore water pressures may have been due to volume increases in the samples which may have taken place during the saturation process. The β' value for the sand samples appears to be consistent with the initial void ratio.

6.4 Discussion of Results - Partly Saturated Series

After the completion of the test series on the saturated compacted samples, an attempt was made to carry out triaxial tests on the partly saturated series with the measurement of pore air and pore water pressures using equipment readily available at the University of Alberta. This test series was carried out with the view to comparing results obtained to the results of the saturated series and thus determine X values for the partly saturated samples. Difficulties in test procedures, required modifications to existing equipment and the limited time available prevented a successful testing program on the partly saturated series from being accomplished. The various phases of the testing program are discussed below.

(a) Evaluation of Test Procedures and Equipment

In order for air pressures in the sample to be measured effectively, it is necessary to have the measuring device as close to the sample as possible since air is a compressible fluid and requires a certain

amount of pressure to compress the air before a pressure in the air will be registered by the measuring device. Due to limitations of the equipment, it was necessary to connect the 30 psia transducer (used for measuring the air pressure) outside the triaxial cell as shown in FIGURE 5 (refer to Section 5.5). The sample was connected to the transducer by a plastic tubing which introduced a volume of air to be compressed before an air pressure would be registered by the transducer. To decrease this volume of compressible air, the plastic tubing and the transducer adapter were filled with a light oil. The sample remained in the triaxial cell for a period of approximately 20 hours prior to actual testing to allow the capillary pressure in the sample to reach its maximum value. It was noted that during this period oil was drawn into the sample. This phenomenon may have been due to one of two causes or a combination of both. The drawing of oil into the sample may have been due to the development of negative air pressure in the sample caused by the release of compaction stresses and by small isolated air bubbles with sub-atmospheric internal pressures (Donald 1963). A second explanation is that a change in temperature in the air phase during the 20 hour period before testing may have induced a negative pressure in the air. It was concluded that oil could not be used in the pore air line and that modifications to the existing equipment were necessary. A suggested modification would be to design a loading cap in which the electrical transducer could be placed next to the sample. This would probably necessitate the use of larger samples since a 1.42 in. diameter loading cap would not allow sufficient area for the suggested modification. It is also obvious that testing should

be carried out in a temperature controlled room to discount the possibility of temperature changes affecting the initial pore air pressure readings.

The fine porous base disk used to transmit pore water pressures had an air entry value or moisture retention value of 14.1 psi. This data was provided by the manufacturer, but equipment was not available to confirm the accuracy of this information and so the value was accepted as true. Cured asphalt samples were set up in the triaxial cell to determine whether the moisture retention value of the porous disk was of sufficient magnitude to prevent water from being drawn into the samples. Water was immediately drawn in through the porous disks into samples and it was concluded from this that the initial negative pore water pressures of the cured samples were in excess of 14.1 psi. Further modifications of the equipment would be necessary to prevent the wetting-up at the bottom of the samples. Suggested modifications would be the acquisition of fine porous disks of a higher moisture retention value. One known source of supply is Aerox Ltd., Scotland, which manufactures porous disks having a moisture retention value of 30 psi. Bishop and Henkel (1962) list suppliers of fine porous disks with moisture retention values of up to 60 psi. Another suggested modification would be the use of a compressed air supply to control the air pressure in the samples and at the same time change the initial pore water pressure to a less negative value at which cavitation of the water in the pore water line does not take place. This method is known as the translation method and has been successfully used on compacted clays by Hilf (1956) and Bishop et al (1960).

A problem also occurred in the determination of the initial volume changes of the samples due to the application of the confining pressures. In order to prevent leakage of the cell water through the piston casing, castor oil was used in the upper portion of the cell fluid as shown in FIGURE 5 (refer to Section 5.5). It was noted that the castor oil contained air bubbles which were impossible to remove without application of pressure. Since volume changes in the partly saturated samples could only be measured by volume changes in the cell water, the compression of air bubbles in the castor oil caused an incorrect volume change in the sample to be recorded. This problem was not encountered in the saturated test series since the cell water was under a pressure of 70 psi before the confining pressure was applied and the air bubbles had been dissolved. It was therefore decided to base the initial volume changes in the samples as that volume change which occurred after confining pressure had been applied for 10 seconds. During this 10 second interval it was assumed that the air bubbles had been dissolved and that the lucite ring had achieved its full expansion. Hence no corrections for volume change in the triaxial test apparatus were made during these tests. This procedure was similar to that used by Schaub and Goetz (1961) in determining volume changes in bituminous mix samples. Volume changes during the application of the axial load were recorded in the usual manner as no error, aside from the membrane effect on volume changes, was thought to be present during this phase of the testing.

The diffusion of air from the sample through the membrane, as

discussed in Section 3.13, was not considered to be a particular problem in this test series since the duration of the actual traixial test was only about one and one half hours.

The necessary modifications to equipment and the acquisition of additional testing equipment could not be accomplished in the limited time available. It was therefore decided to carry out only one test series measuring the pore water and pore air pressures, remembering the limitations of the equipment. This test series was carried out on the compacted sand samples. The remaining test series were carried out with the object of measuring the strength characteristics of the samples in terms of total stresses and the volume changes in the samples during application of axial loads.

(b) Stress Lines From Modified Mohr Envelopes

FIGURE 11 shows the results obtained for triaxial tests on the partly saturated sand samples (Series UA). Also superimposed on this figure is the effective stress line obtained for the saturated samples (Series SA). The two stress lines for the partly saturated sand samples take into account the measured pore water and pore air pressures. A calculation of X values by the method outlined in Section 3.12, using the stress lines shown in FIGURE 11, would result in negative values for X . According to available literature, X values are always positive and vary between 0 and 1. From an evaluation of testing procedures and equipment, it was suspected that correct pore air pressures were not recorded. Utilizing Equation 6 (refer to Section 3.4), an idea

was obtained of the magnitude of pore air pressures which were to be expected from the measured volume changes in the samples. Appendix D shows the calculated values of pore air pressures for the sand samples and are compared to the corresponding measured pore air pressures. The calculated pore air pressures were 10 to 20 times greater than the measured values. It was concluded that pore air pressures were not being measured efficiently and also that incorrect pore water pressures were being measured, since the pore air pressures would affect the magnitude of the pore water pressures. Because an acceptable procedure could not be devised for measuring pore air pressures with the equipment available, it was considered inadvisable to continue with the measurement of pore air and pore water pressures in the succeeding test series. It was thus decided to carry out the remaining test series with the object of determining strength characteristics in terms of total stresses.

FIGURE 12 shows the total stress lines obtained for the cured asphalt samples and soaked foamed asphalt samples (Series UB, UC and UD). It is evident from these results that the cured foamed asphalt samples possessed superior strength characteristics compared to the cured wet hot mix samples. It is also evident that the total strengths of the cured foamed asphalt samples were considerably reduced upon soaking. These findings are similar to those of Haas (1963) and Laplante (1963) and serve to support their experimental results.

The difference in strength of the two types of cured asphalt samples lies mainly in the difference between the cohesion (d) values,

as the β values differ by only $.8^{\circ}$. The difference in d values is 13 psi whereas comparison to the d' values obtained for the saturated series shows a difference of only 2 psi. It is postulated that the difference in d values for the two types of cured asphalt samples is due to a combination of better asphalt distribution throughout the foamed asphalt samples and the greater effect of the interfacial tensions between phases on the viscosity of the asphalt films.

Soaking of the cured foamed asphalt samples resulted in a considerable strength loss in terms of total stresses. It is postulated that this strength loss was due to a reduction in the interfacial tensions between phases, resulting in a weakening of the tensions set up in the asphalt films and water phase upon curing. Also the fact that the samples increased in volume during soaking would cause a decrease in the strength characteristics of the samples.

Based on the above reasoning, a decrease in moisture content of asphalt stabilized samples would result in increased capillary pressures in the water phase and increased viscosity of the asphalt phase. This increased viscosity would result in an increased effective cohesion intercept at a particular moisture content. If the above postulations are true, then the method of determining X values as outlined for partly saturated soils may not be applicable to asphalt stabilized soils.

These postulations could not be proved experimentally in this testing program due to limitations of testing equipment. Although a fairly sophisticated test set up was used in this testing program it is obvious that additional equipment and modifications to existing equip-

ment are required to prove or disprove the postulations made in this section. It is also suggested that a detailed study of the physical chemical aspects of an asphalt-stabilized soil compared to those of the untreated samples is in order.

(c) Stress-Strain Curves

Typical stress-strain curves for the partly saturated samples, at a confining pressure of 20 psi, are shown in FIGURE 13. Shown in conjunction with these curves are the corresponding volume changes which occurred in the samples during shearing.

The stress-strain curves reveal the same findings as were noted in the modified Mohr envelopes for the partly saturated samples. The cured foamed asphalt sample displayed the highest deviator stresses which were considerably reduced upon soaking. The cured wet hot mix sample also displayed high deviator stresses but were less than those obtained for the cured foamed asphalt sample. A more gradual drop off of deviator stresses was noted for the wet hot mix sample when compared to the foamed asphalt sample. This may have been due to the combined effect of better asphalt distribution and increased viscosity in the asphalt phase of the foamed asphalt sample, thus resulting in a more brittle failure. The high deviator stresses of the cured asphalt stabilized samples are believed to be due to the high negative pressures in the pore water phase and the increased viscosity in the asphalt phase. The soaked foamed asphalt sample displayed lower deviator stress values as compared to the sand sample due mainly to the volume change incurred

during soaking and the resulting lower dry unit weight of the sample.

FIGURE 13 also shows the dilatant characteristics of the cured asphalt stabilized samples at low strains, whereas the dilatancy in the soaked sample was negligible. Volume change readings were probably influenced by the membrane effect as outlined in Section 3.13 and may have accounted for greater volume decreases than actually occurred in the samples. The recorded volume changes for the two types of cured asphalt stabilized samples were approximately the same whereas the deviator stress varied greatly. This was generally true for all the cured samples tested and would further suggest that the greater deviator stress may be due to the better distribution of asphalt and the higher viscous effects of the asphalt films in the cured foamed asphalt stabilized samples.

(d) Initial Capillary Pressures in Various Samples

It is noted from the outline of the testing program given in Section 5.7 that Sample Nos. 26 and 30 were not sheared. These samples plus a compacted sand sample, a compacted wet hot mix stabilized sand sample and a soaked cured foamed asphalt stabilized sand sample were used to investigate the initial capillary pressures in the various samples. The capillary pressures were determined by the difference between the measured pore air and pore water pressures over a 20 hour period. The results obtained are shown in FIGURE 14. It is believed however that the pore air pressure readings were affected by the volume of air between the sample and the measuring device and by temperature changes that may have taken place over the 20 hour period. It is therefore questionable

whether the initial capillary pressure curves shown in FIGURE 14 represent the true initial capillary pressures in the samples. The capillary pressure curves shown for the cured asphalt sand samples are definitely not true since considerable wetting-up at the bottom of the samples occurred. Due to the uncertainty of the testing procedures and equipment, the results shown will not be discussed fully.

It is to be noted that the capillary pressures in the enclosed samples increased with time. This finding was similar to that of Bishop (1960) and shows that a period of time is required for the capillary pressures to reach their maximum values. The maximum capillary pressure for the compacted wet hot mix sample was about 50 per cent of that for the compacted sand samples. This shows the effect of the presence of asphalt on the reduction of capillary pressures. The soaked cured foamed asphalt sample showed an even lower maximum capillary pressure due to the fact that its "thirst" for additional water had been largely satisfied during the soaking period. The wetting-up at the bottom of the cured samples meant that the initial capillary pressures of these samples were in excess of the moisture retention value of the porous base plates.

In order to ensure that accurate initial capillary pressures are being measured, it is necessary that the modifications to the triaxial test equipment, mentioned in the (a) part of this section, be carried out.

6.5 Mohr Rupture Envelopes

The results for various test series in the testing program are summarized in the form of the common Mohr rupture envelopes as shown in FIGURE 15. The Mohr envelopes were derived from the results of the modified Mohr envelopes, which have been previously discussed, by utilizing Equation 8 (refer to Section 3.5). The Mohr envelopes are presented in terms of effective stresses for the saturated series and in terms of total stresses for the partly saturated series. The effective stress lines shown for the asphalt stabilized sand samples are the average of the stress lines obtained from the as-compacted, cured and soaked test series, since very little difference was evident in the results obtained for the modified Mohr envelopes.

It is apparent from FIGURES 15 that the shear strength of the asphalt stabilized sand, in terms of effective stresses, was superior to the shear strength of the untreated sand only up to an effective normal stress of 27 psi after which the shear strength of the sand samples was greater than that of the asphalt stabilized sand samples. The cohesion intercept for the foamed asphalt samples was 7.3 psi and the effective angle of internal friction was 34.1^{o}. The cohesion intercept for the wet hot mix samples was 5.0 psi and the effective angle of internal friction was 36.1^{o}. As already discussed in Section 6.3, it is believed these differences were due to a better asphalt distribution in the foamed asphalt samples and the effect of interfacial forces between the water and asphalt phases on the viscosity of the asphalt films.

The total strength envelopes for the cured asphalt samples show the superior strength characteristics of the foamed asphalt samples over the wet hot mix samples. The foamed asphalt series displayed a cohesion intercept of 48.8 psi and an angle of internal friction of 32.7° while the wet hot mix series showed a cohesion value of 34.3 psi and an angle of internal friction of 34.3°.

As postulated in Section 6.4, the cohesion difference is believed to be due to a combination of a better distribution of asphalt and the resulting greater effect of curing on the viscous characteristics of the asphalt films in the foamed asphalt samples. The large cohesion intercepts displayed by the cured samples were probably due to a combination of capillary pressures in the water phase and the increased viscosity of the asphalt phase.

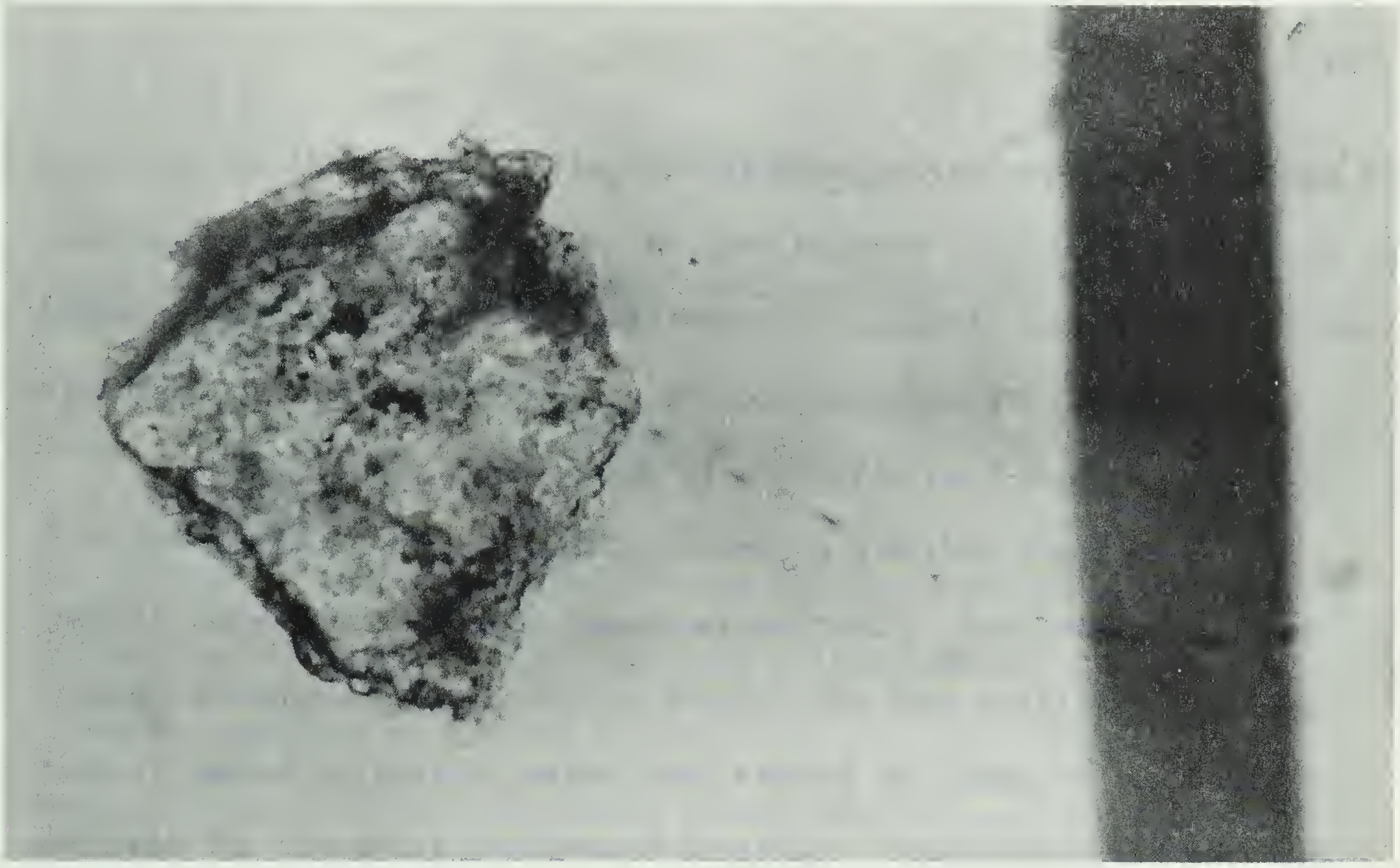
6.6 Microscopic Examination of Sand-Asphalt Mixes

Throughout this investigation it was assumed that the foamed asphalt method of preparation produced a better distribution of asphalt throughout and around the sand particles than did the wet hot mix method of preparation. At the conclusion of the testing program it was decided to carry out a microscopic examination of the mixes produced by the two methods of preparation. The microscopic examination of sample mixes was carried out using a Vickers M 15 A polarizing microscope with a 35 mm camera attachment. The pictures shown in PLATES 2 and 3 are representative of the mixes viewed. The grids shown in PLATE 2 represent a 1 square millimeter grid.

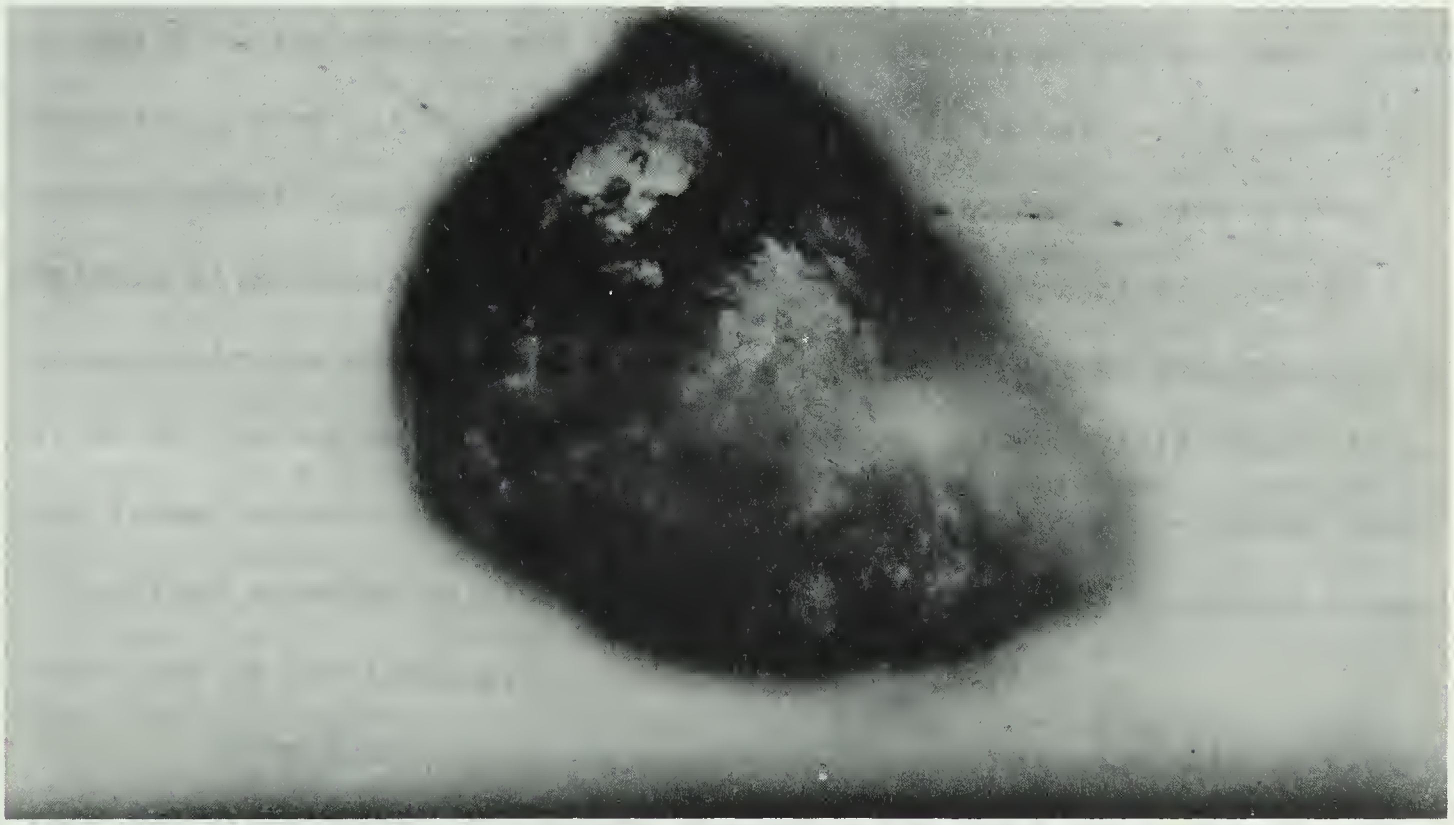
(A) SAND-WET HOT MIX PARTICLES

(B) SAND-FOAMED ASPHALT PARTICLES

PLATE 2 - MICROSCOPIC PICTURES OF SAND-ASPHALT MIXTURES, MAGNIFICATION x 130



(A) SAND-WET HOT MIX PARTICLE



(B) SAND-FOAMED ASPHALT PARTICLE

PLATE 3 - MICROSCOPIC PICTURES OF ISOLATED SAND - ASPHALT PARTICLE, MAGNIFICATION x 470

To the naked eye the distribution of the asphalt throughout the sand appeared to be quite lean for both methods of preparation. This was probably due to the fact that the amount of asphalt was low of optimum although optimum asphalt content was not determined.

Microscopic examination of the cured sand-asphalt mixes, at 30 times magnification, generally revealed a better distribution of asphalt throughout the sand-foamed asphalt mix. Representative sampling showed that approximately 65 per cent of the sand particles were completely coated or partly coated with asphalt as compared to about 45 per cent for the wet hot mix-sand sampling. The microscopic pictures shown in PLATE 3 reveal spotty distribution on the asphalt-sand particle prepared by the wet hot mix method whereas the foamed asphalt-sand particle displays a more uniform and complete coating of asphalt. The coated foamed asphalt-sand particles shown in PLATE 2 appear to have a more uniform film thickness than the wet hot mix sand particles. This is revealed in the photographs by a mixture of transluscent and opaque films on the wet hot mix-sand particles which was generally absent in the foamed asphalt-sand mix. Due to limitations of the microscope used, a detailed examination of the asphalt-sand samples in a compacted state could not be carried out.

From these microscopic examinations it was generally concluded that the distribution of asphalt throughout the sand was better for the foamed asphalt method of preparation than for the wet hot mix process and that a more uniform film thickness resulted from the foamed asphalt process of sample preparation.

CHAPTER VII

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

The main purpose of this thesis was to investigate the shear strength of an asphalt stabilized sand utilizing the effective stress equations applicable to saturated and partly saturated soils. The conclusions reached from this preliminary investigation were drawn from a fairly comprehensive literature review and a triaxial test program which was carried out to evaluate two different methods of sample preparation. The conclusions presented are limited by the scope of the testing program in that only one soil type and one type of asphalt were used, as many variables as possible were made constant and arbitrary test conditions such as curing and soaking, which do not necessarily simulate field conditions, were utilized.

The following conclusions were made from the literature review and the results of the testing program. The conclusions drawn from the testing program are applicable only to the particular mixtures used and the testing techniques employed.

(1) From the literature review it is concluded that a shear strength analysis of an asphalt stabilized sand, in terms of effective stresses, is a complex problem as the interplay of surface tension forces with the viscosity of the asphalt may affect

the strength characteristics of the sand-asphalt mix.

(2) From the microscopic examination of the sand-asphalt mixes it is concluded that the foamed asphalt method of preparation provides a better distribution throughout, and a more uniform coating of, the sand particles than does the wet hot mix method of preparation.

(3) From the saturated test series the following conclusions are made:

(a) Saturation of the compacted samples using the back pressure method may result in a volume increase in the samples.

(b) The foamed asphalt stabilized sand mix displays a higher cohesion in terms of effective stresses but a slightly lower angle of internal friction as compared to the wet hot mix stabilized sand. This is believed to be due to the better and more uniform distribution of the asphalt throughout the sand resulting in a better bonding between particles.

(c) The curing and soaking conditions do not induce a permanent change in the strength characteristics, in terms of effective stresses, of the asphalt stabilized sand samples.

(d) The asphalt stabilized sand mixes show superior shear strength, in terms of effective stresses, up to an ef-

fective normal stress of 27 psi. This is due to the in- increase in cohesion and the resulting decrease in the angle of internal friction caused by the asphalt films around the sand particles.

(e) The pore water pressure curves obtained from triaxial tests on the saturated samples are inconsistent with published results for sand and sand-asphalt mixes in a dense or medium dense state. This inconsistency may have been due to volume changes which may have taken place during saturation of the compacted samples.

(4) The following conclusions are made from the triaxial testing of the partly saturated series:

(a) The testing apparatus used for the partly saturated test series is not adequate for obtaining accurate test results for an analysis of the shear strength in terms of effective stresses.

(b) The cured foamed asphalt stabilized sand displays greater shear strength in terms of total stresses as compared to the wet hot mix stabilized sand. It is believed that this may be due to the better and more uniform distribution of asphalt throughout the sand and to the increased effect on the viscosity due to the curing conditions.

(c) The shear strength of the foamed asphalt stabilized sand,

in terms of total stresses, is considerably reduced upon soaking due to the release of capillary pressures in the water phase and also, it is believed, to the decrease in the viscosity in the asphalt phase.

(5) From the literature review and the results of the saturated and partly saturated test series it is concluded that the method of determining values of the factor X for soils may not be applicable to the asphalt stabilized sand. This conclusion is based mainly on the conjecture that the effective stress line obtained from triaxial tests on saturated asphalt stabilized sand samples does not represent the true effective stress line for the partly saturated samples. This conclusion requires further experimental evidence to prove or disprove its validity.

7.2 Recommendations

This preliminary investigation has revealed that further research is required in order to arrive at definite conclusions concerning the shear strength of an asphalt stabilized soil in terms of effective stresses. Future investigation should be concentrated on developing definite test procedures to be followed in analyzing the effective stresses of an asphalt stabilized soil in a partly saturated condition. The re-recommendations listed below apply to modifications to existing test equipment, to additional equipment that must be acquired and to future investigations into the effective stresses of an asphalt stabilized soil.

(1) The following modifications to existing test equipment and test procedures are recommended:

(a) Design of a loading cap in which the electrical transducer can be placed for measurement of air pressures in the partly saturated samples. This modification will probably necessitate the use of 2 to 2 1/2 inch diameter samples.

(b) The use of a controlled air pressure in the partly saturated samples to prevent the wetting-up of samples and to translate initial negative pore water pressures to a measurable magnitude.

(c) The use of a volume change indicator which will permit detection of volume changes in the samples during the saturation process. The original volumes of the samples can then be maintained during saturation of the samples.

(2) It is recommended that the following additional equipment be acquired:

(a) Fine porous base disks with a moisture retention value or an air entry value of sufficient magnitude to prevent wetting-up at the base of the samples.

(b) If triaxial tests are to last in excess of about one and one-half hours, then a triaxial cell capable of using mercury as the cell fluid must be acquired to prevent diffusion of air from the sample through the membrane.

(3) It is recommended that test procedures for analyzing effective stresses of partly saturated samples be first developed using soil only. Once the testing procedure is established for soils, then the procedures can be applied to asphalt stabilized soils.

(4) It is recommended that curing and soaking conditions similar to actual field conditions be used in future testing programs. This would require field testing to determine degrees of saturation under particular environmental conditions.

(5) In addition to achieving acceptable testing procedures, it is recommended that a detailed study of the physical chemical aspects of an asphalt stabilized soil be carried out. Particular emphasis should be placed on the effect of the interplay of surface tensions in the various phases on the viscosity of the asphalt film of an asphalt stabilized soil.

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APPENDIX A

SAND CLASSIFICATION DATA

Specific Gravity Test
Sieve Analysis
Hydrometer Test
Atterberg Limits
Compaction Test

UNIVERSITY of ALBERTA DEP'T. of CIVIL ENGINEERING SOIL MECHANICS LABORATORY **SPECIFIC GRAVITY**	
PROJECT	*Thesis*
SITE	*U of A*
SAMPLE	*Braeborn Fines*
LOCATION	
HOLE	DEPTH
TECHNICIAN *DTC.*	DATE *23 Jun 66*

Sample No.	1	2
Flask No.	DC	DC
Method of Air Removal	Vacuum	Vacuum
W_{b+w+s}	743.12	745.61
Temperature T	25.6 °C	25.9 °C
W_{b+w}	684.40	684.36
Evaporating Dish No.	1	2
Wt. Sample Dry + Dish	238.56	170.78
Tare Dish	144.53	72.72
W_s	94.03	98.06
G_s	2.667	2.668

W_{b+w+s} = Weight of flask + water + sample at T°.

W_{b+w} = Weight of flask + water at T° (flask calibration curve).

W_s = Weight of dry soil

G_s = Specific gravity of soil particles $= \dfrac{W_s}{W_s + W_{b+w} - W_{b+w+s}}$

Determination of W_s from wet soil sample:

Sample No.			Sample No.		
Container No.			Container No.		
Wt. Sample Wet + Tare			Wt. Test Sample Wet + Tare		
Wt. Sample Dry + Tare			Tare Container		
Wt. Water			Wt. Test Sample Wet		
Tare Container			W_s		
Wt. of Dry Soil					
Moisture Content w %					

Description of Sample:

Remarks: *Approximately 100 grams of air dried sand was used in the specific gravity determinations. Deairing was accomplished by vacuum for 20 minutes.*

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SOIL MECHANICS LABORATORY

SIEVE ANALYSIS

PROJECT Thesis
SITE U. of A.
SAMPLE Braeborn Fines
LOCATION
HOLE DEPTH
TECHNICIAN DTC DATE 20 Jun 66

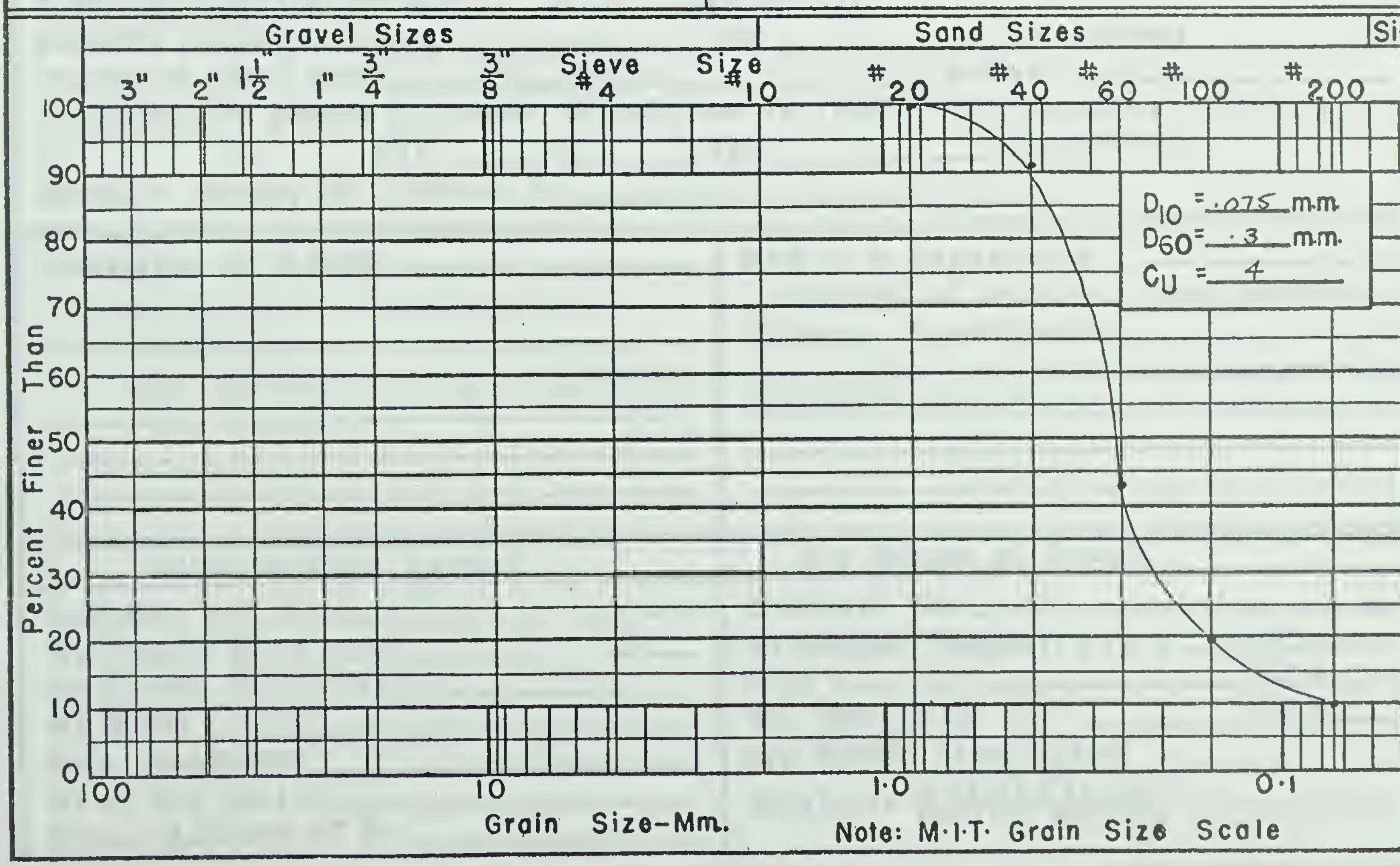
Total Dry Weight of Sample ____	Sieve No.	Size of Opening Inches	Size of Opening Mm.	Weight Retained gms.	Total Wt. Finer Than gms.	Percent Finer Than	% Finer Than Basis Orig. Sample
Initial Dry Weight Retained No. 4							
Tare No. ____							
Wt. Dry + Tare ____							
Tare ____		3/4	19·10				
Wt. Dry ____		3/8	9·52				
	4	·185	4·76	0	500	100.0	
Passing	4						
Initial Dry Weight Passing No. 4	10	·079	2·000				
Tare No. ____	20	·0331	·840	.7	499.3	99.8	
Wt. Dry + Tare ____	40	·0165	·420	44.9	454.4	91.2	
Tare ____	60	·0097	·250	237.4	217.0	43.4	
Wt. Dry ____	100	·0059	·149	119.3	97.7	19.5	
	200	·0029	·074	48.3	49.4	9.9	
Passing	200						

Description of Sample ____

Time of Sieving ____

Method of Preparation ____

Remarks Air dried sample washed through nest of sieves, then oven dried.



UNIVERSITY of ALBERTA
DEP'T· of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
HYDROMETER TEST

PROJECT Thesis
SITE
SAMPLE Braeburn Fines
LOCATION
HOLE DEPTH
TECHNICIAN DATE 31 Aug 66

Date	Temp·	Time	Elapsed Time	R'_h	$R_h = R'_h + c_m$	D m.m.	$R_h + m_t - c_d$	W %	W % Basis Orig. Sample	Remarks
27 Aug.	26°C	1556	0							
	26		15 sec	24.5	25.0	.0875	25.0	65.2	12.5	
	26		30 sec	20.0	20.5	.0650	20.0	49.6	9.9	
	26		45 sec	19.0	19.5	.0540	19.5	45.0	9.0	
	26		60 sec	17.0	17.5	.0475	17.5	40.5	8.1	
	26		1.5 min	15.3	15.8	.0400	15.8	39.4	7.8	
	26		2 min	14.3	14.8	.0355	14.8	37.9	7.5	
	26	1600	4 min	12.9	13.4	.0245	13.4	35.4	7.0	
	26	1604	8 min	11.8	12.3	.0178	12.3	32.8	6.5	
	26	1611	15 min	10.0	10.5	.0131	10.5	27.3	5.4	
	25.6	1626	30 min	9.6	10.1	.0096	10.0	25.3	5.0	
	24.8	1900	184 min	7.0	7.5	.0039	7.2	24.8	4.9	
28 Aug	23.8	1200	1204 min	5.0	5.5	.0016	5.0	12.6	2.5	
29 Aug	24.0	905	2469 min	4.5	5.0	.0011	4.5	7.6	1.5	

Hydrometer No.s 9853 R and ______ Graduate No. ______

$W\% = \frac{100}{W_s} \cdot \frac{S_s}{S_s - 1}(R_h + m_t - c_d) =$ 3.22 $(R_h + m_t - c_d)$

Meniscus correction $= c_m =$ 0.5 and ______ respectively

Dispersing agent used 6% Calgon Amount 25 ml.

Correction for change in density of liquid due to addition of dispersing agent $= c_d$

$c_d =$ 1.3 and ______ respectively

Specific Gravity of Solids $= G_s$ 2.67

Description of Sample ______

Method of Preparation 50 grams of oven dry sand passing through No. 100 sieve.

Remarks ______

Initial Moisture Content

Container No. ______
Wt. Sample Wet + Tare ______
Wt. Sample Dry + Tare ______
Wt. Water ______
Tare Container ______
Wt. of Dry Soil ______
Initial Moisture w % ______

Dry Weight of Sample

Container No. ______
Wt. Sample (~~Wet~~/Dry) + Tare 1033.0
Tare 983.3
Wt. (~~Wet~~/Dry) Soil 49.7
Dry Weight from Initial Moisture $= \frac{100 \times \text{Wt. Wet Soil}}{100 + \text{Init. Moist.}\%} =$ ______

A4

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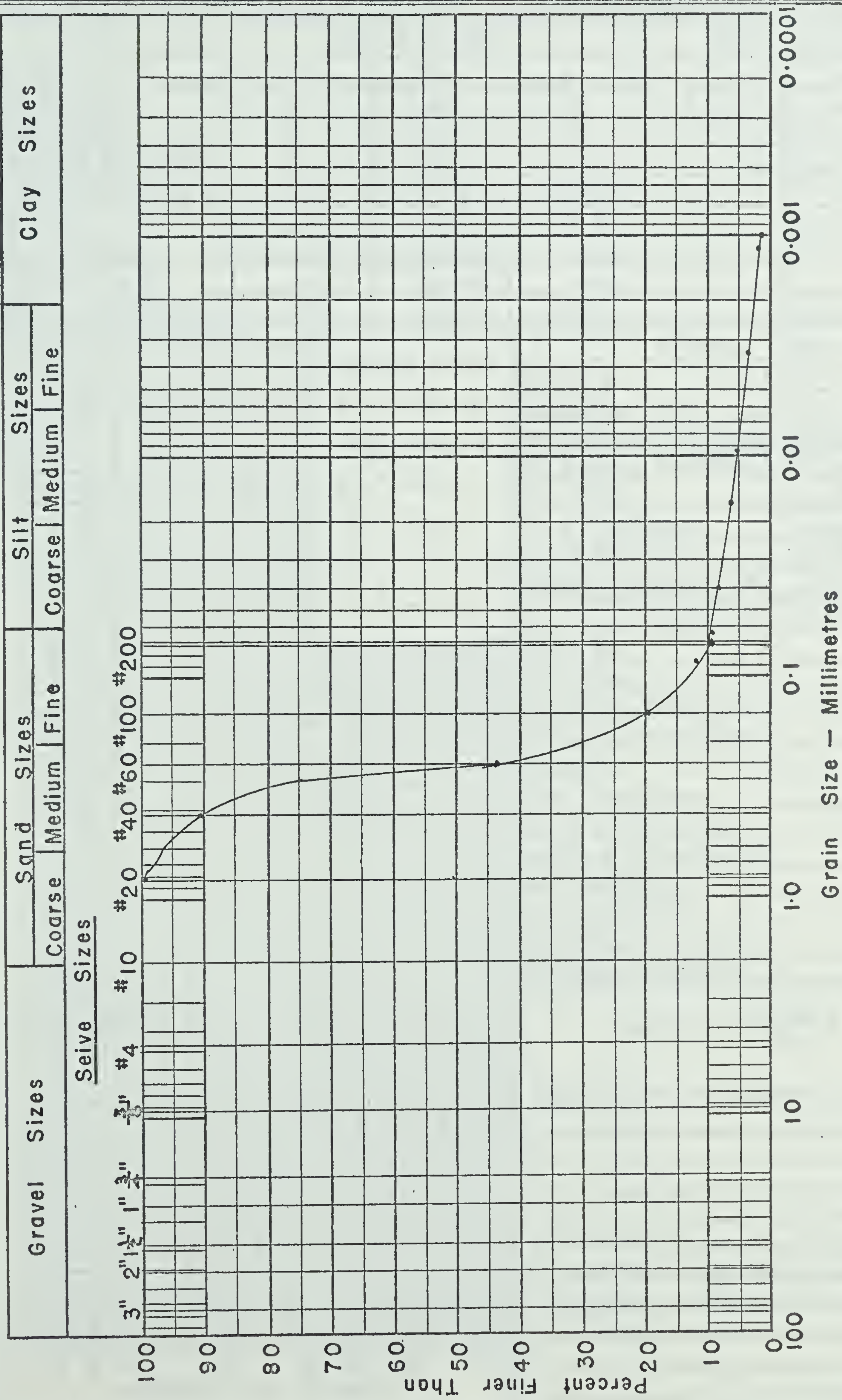
DEP T. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

GRAIN SIZE CURVE

PROJECT *Thesis*
SITE
SAMPLE *Braeborn Fines*
LOCATION
HOLE DEPTH
TECHNICIAN DATE *31 Aug 66*



D_{10} = ____ mm.
D_{60} = ____ mm.
C_U ____

Remarks:

Note: M.I.T. Grain Size Scale

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SOIL MECHANICS LABORATORY

ATTERBERG LIMITS

A5

PROJECT	Thesis
SITE	U of A
SAMPLE	Braeborn Fines
LOCATION	
HOLE	DEPTH
TECHNICIAN DTC.	DATE 20 Jun 66

Liquid Limit				W_L using one point method		
Trial No.	1	2	3	1	2	3
No. of Blows	24	23	14	25.7%	26.0%	26.3%
Container No.	6	7	8			
Wt. Sample Wet + Tare	43.36	49.68	47.55		Average W_L =	26.0%
Wt. Sample Dry + Tare	40.46	45.81	43.94			
Wt. Water	2.90	3.87	3.61			
Tare Container	29.16	30.86	30.96			
Wt. of Dry Soil	11.30	14.95	12.98			
Moisture Content w%	25.7	26.1	27.8			

Average Values

w_l = 26%

w_p = 26.5%

w_s =

I_p = 0

I_f =

I_t =

Moisture Content %

7 8 9 10 15 20 25 30 40

Number of Blows

Plastic Limit			
Trial No.	1	2	3
Container No.	8	9	10
Wt. Sample Wet+Tare	42.29	47.08	45.39
Wt. Sample Dry+Tare	39.93	44.34	42.72
Wt. Water	2.36	2.74	2.67
Tare Container	30.64	34.32	32.87
Wt. of Dry Soil	9.29	10.02	9.85
Moisture Content %	25.4	27.2	27.0

Shrinkage Limit			
Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V-V_o$			
Shrinkage Limit w_s			

$$w_s = w - \left(\frac{V-V_o}{W_o} \times 100\right)$$

Description of Sample:

Remarks: One point method used in determining the liquid limit W_L, using $\tan\beta = .1$

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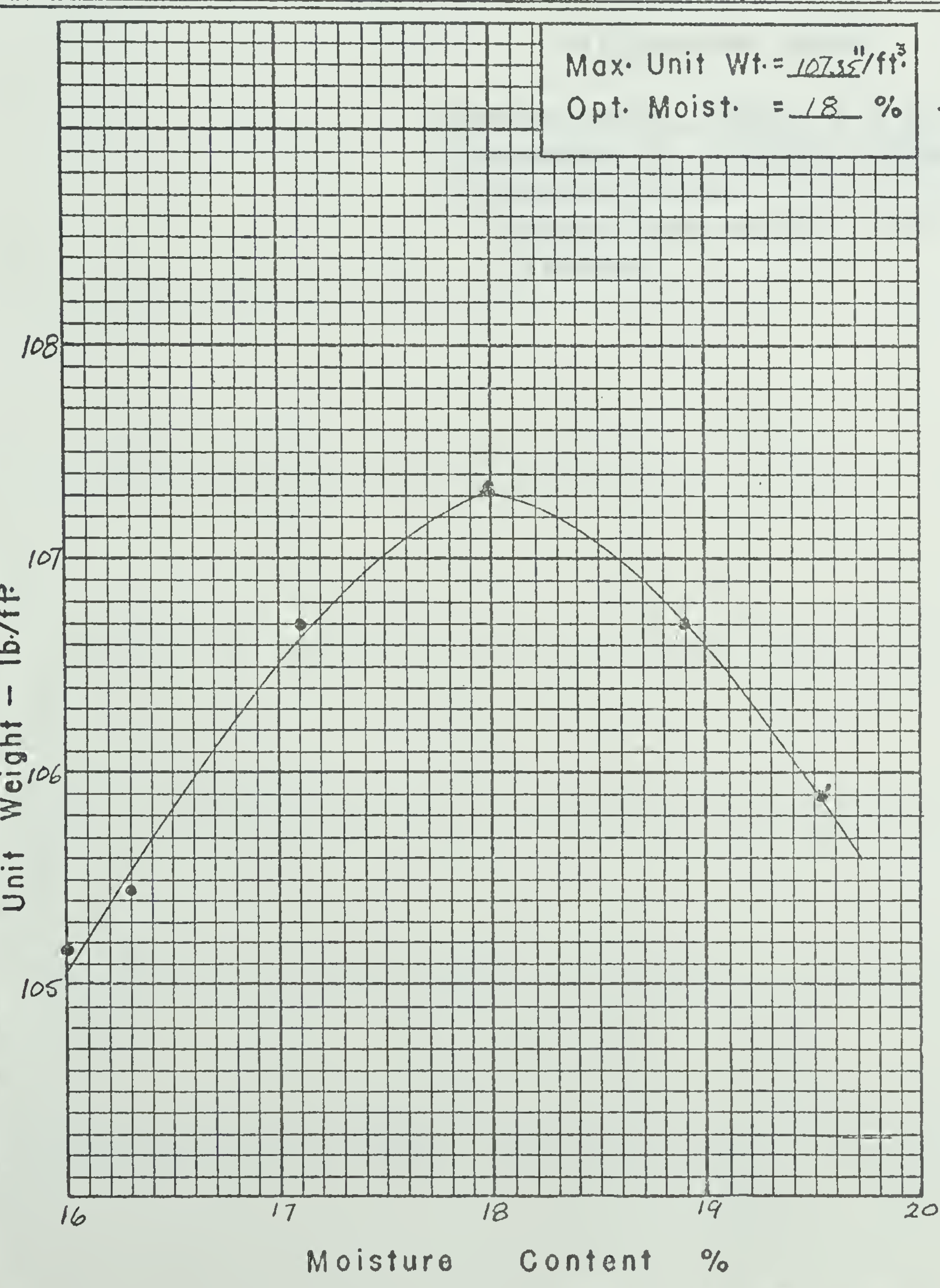
SOIL MECHANICS LABORATORY

COMPACTION TEST

PROJECT Thesis
SITE U of A
SAMPLE Braeborn Fines
LOCATION
HOLE DEPTH
TECHNICIAN DTC DATE 25 Jun 66

Trial Number							
Unit Weight Determination — Mold No·	1	1	1	1	1	1	
Wt· Sample Wet + Mold	1821.3	1826.5	1826.5	1826.8	1821.9	1824.6	
Wt· Mold	1681.8	1681.8	1681.9	1681.8	1681.7	1681.8	
Wt· Sample Wet	139.5	144.7	144.6	145.0	140.2	142.8	
Volume Mold	.00252	.00252	.00252	.00252	.00252	.00252	
Wet Unit Weight lb·/ft^3	122.02	126.58	126.55	126.85	122.65	124.95	
Dry Unit Weight lb·/ft^3	105.19	107.36	105.90	106.69	105.46	106.70	
Moisture Content Determination — Container No·	93	83	71	72	38	69	
Wt· Sample Wet + Tare	168.0	173.9	168.4	166.1	210.3	165.6	
Wt· Sample Dry + Tare	149.0	152.1	144.5	143.2	190.9	144.8	
Wt· Water	19.0	21.8	23.9	22.9	19.4	20.8	
Tare Container	29.9	30.6	22.2	21.9	71.9	23.0	
Wt· Dry Soil	119.1	121.5	122.3	121.3	119.0	121.8	
Moisture Content	15.95	17.94	19.54	18.88	16.30	17.10	

Max· Unit Wt· = 107.35 lb/ft^3
Opt· Moist· = 18 %



Method of Compaction
Kneading compaction

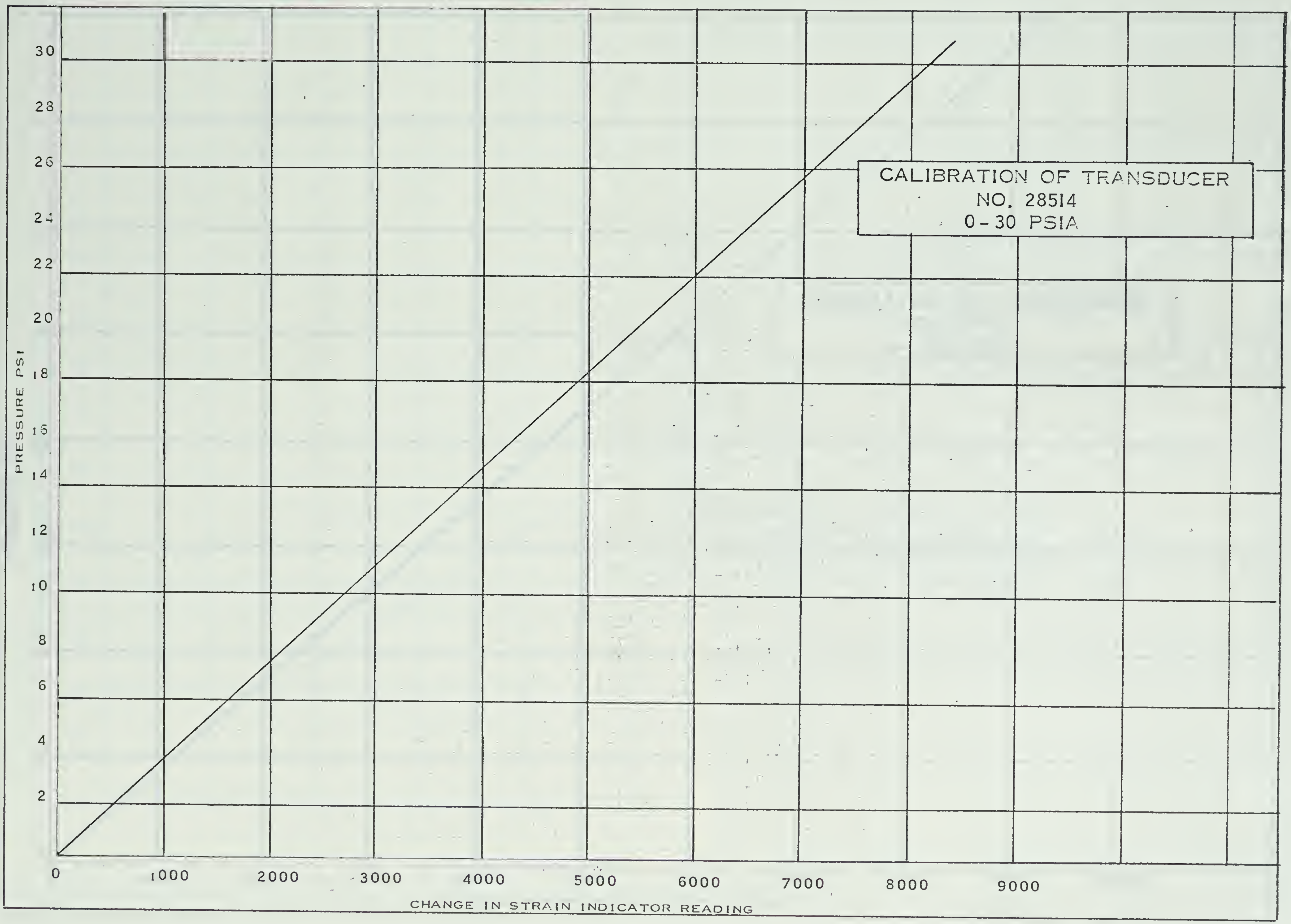
Diam. Mold 1.42 in.
Height Mold 2.84 in.
Volume Mold .00252 ft^3
No· of Layers 3
Blows per Layer 25
Ht· of Free Fall
Wt· of Tamper 20 lb. spring
Shape of Tamping Face Circular
Description of Sample

Remarks

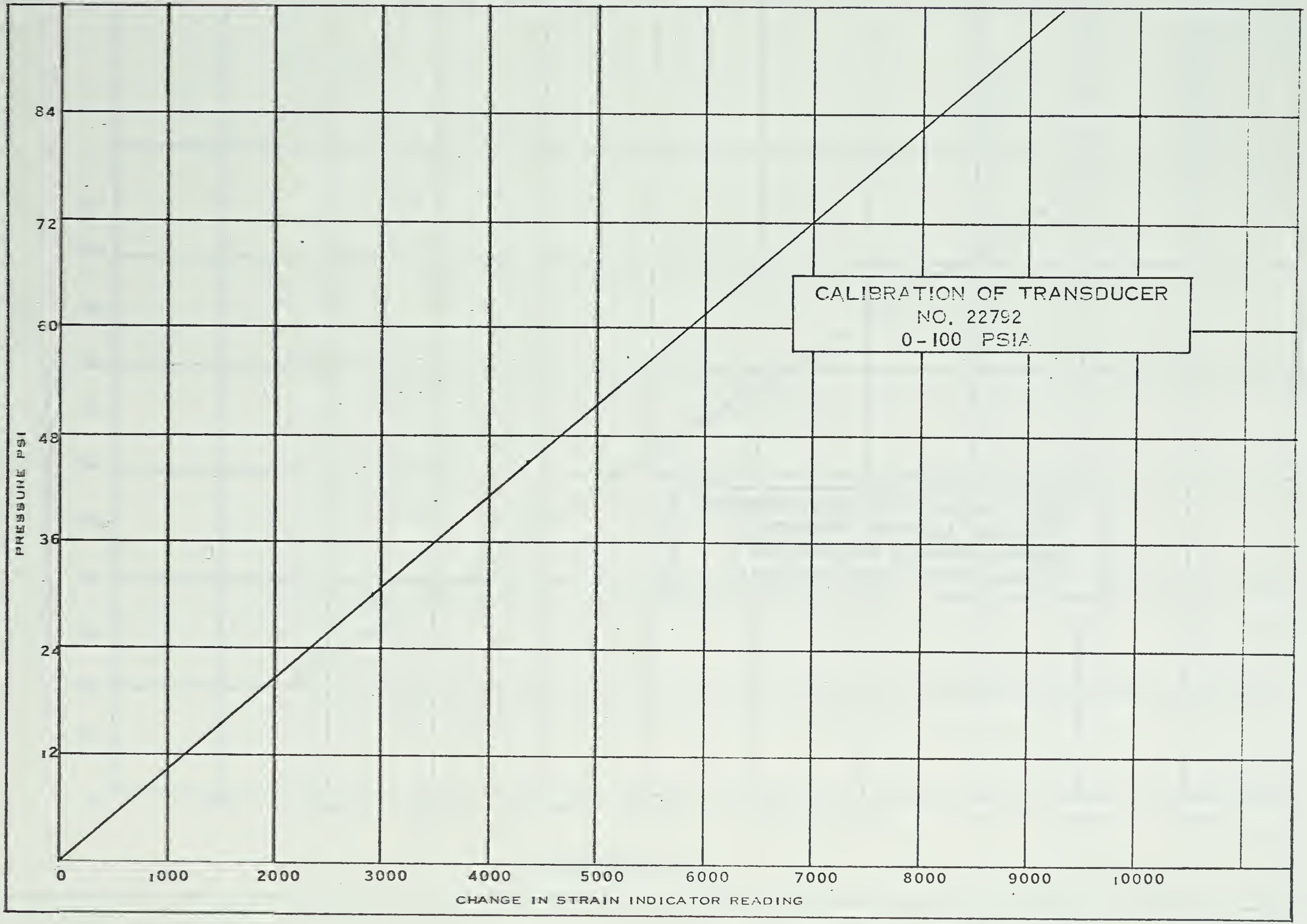
APPENDIX B

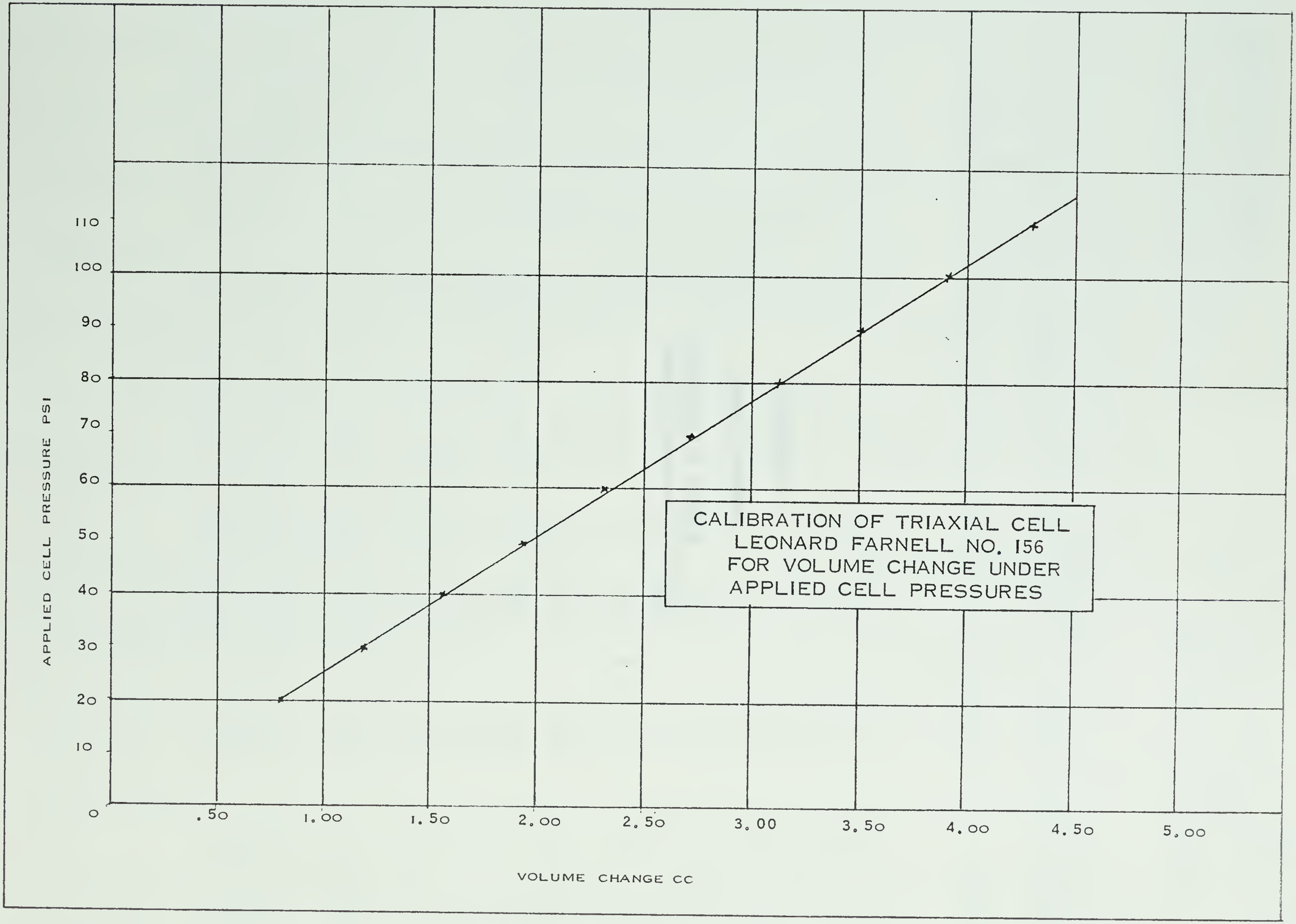
CALIBRATION CURVES

Transducer No. 28514 0-30psia
Transducer No. 22792 0-100psia
Triaxial Cell No. 156 for Volume
Change Under Applied Cell
Pressure



CALIBRATION OF TRANSDUCER
NO. 28514
0 - 30 PSIA
PRESSURE PSI
30
28
26
24
22
20
18
16
14
12
10
8
6
4
2
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
CHANGE IN STRAIN INDICATOR READING


CALIBRATION OF TRANSDUCER
NO. 22792
0-100 PSIA
PRESSURE PSI
84
72
60
48
36
24
12
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
CHANGE IN STRAIN INDICATOR READING


CALIBRATION OF TRIAXIAL CELL
LEONARD FARNELL NO. 156
FOR VOLUME CHANGE UNDER
APPLIED CELL PRESSURES
APPLIED CELL PRESSURE PSI
110
100
90
80
70
60
50
40
30
20
10
0
.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
5.00
VOLUME CHANGE CC

APPENDIX C

SAMPLE SHEETS

Sample Data Sheets
Sample Calculation Sheets

UNIVERSITY OF ALBERTA
Department of Civil Engineering

Project Thesis

Triaxial Compression Test on Compacted Asphalt Stabilized Soil

Sample Nos. 12 8 Dec. 66

Str. Dial Div. .001	Str. %	No. of Stress Dial Div.	Load lbs.	Vol. Change Indic. Rdg.	Vol. Change	Corr. Area in^2	Deviator Stress $\sigma_1 - \sigma_3$	Str. Indic. Rdg.		Pore Pressure		Effective Stress psi	
								u_w	u_a	u_w	u_a	σ'_1	σ'_3
		.0001											
0	0	0	0	4.19	0	1.59	0	2161	—	0	—	0	20
10	.35	116	86	4.09		1.60	53.75	1489		3.5		70.25	16.5
20	.71	126	92	4.06		1.60	57.50	1288		9.0		68.50	11.0
30	1.06	132	95	4.03		1.61	59.01	1212		9.7		69.04	10.3
40	1.41	135	98	4.00		1.61	60.87	1190		10.0		70.87	10.0
50	1.77	137	99	3.98		1.62	61.11	1189		10.0		71.11	10.0
60	2.12	139	100	3.95		1.62	61.72	1201		9.9		71.82	10.1
70	2.47	141	101	3.92		1.63	61.96	1215		9.8		72.16	10.2
80	2.83	143	102	3.89	.30	1.64	62.20	1231		9.6		72.60	10.4
90	3.18	144	103	3.86		1.64	62.81	1249		9.5		73.31	10.5
100	3.53	145	104	3.85		1.65	63.03	1265		9.3		73.73	10.7
120	4.24	147	105	3.77		1.66	63.25	1291		9.0		74.25	11.0
140	4.95	149.5	108	3.71		1.67	64.67	1322		8.7		75.97	11.3
160	5.65	150	109	3.64		1.68	64.88	1348		8.5		76.38	11.5
180	6.36	151	110	3.58	.61	1.70	64.71	1369		8.2		76.51	11.8
200	7.07	152.5	111	3.52		1.71	64.91	1390		8.0		76.91	12.0
220	7.77	153.5	112	3.46		1.73	64.73	1409		7.8		76.94	12.2
240	8.48	154.5	113	3.39		1.74	64.93	1424		7.6		77.34	12.4
260	9.19	155.5	114	3.36		1.75	65.14	1441		7.5		77.64	12.5
280	9.89	157	115	3.27	.92	1.76	65.34	1459		7.3		78.04	12.7
300	10.60	157.5	116	3.21		1.78	64.61	1475		7.1		77.51	12.9
320	11.31	158.5	116	3.15		1.79	64.80	1491		7.0		77.80	13.0
340	12.01	159	116	3.09		1.81	64.09	—		—		—	—
360	12.72	159	116	3.03		1.82	63.74	1518		6.8		76.94	13.2
380	13.43	159.5	117	2.97		1.83	63.38	1529		6.6		76.78	13.4
400	14.14	160	117	2.91		1.85	63.24	1539		6.5		76.74	13.5
420	14.85	160.5	117	2.84	1.35	1.86	62.91	1550		6.4		76.51	13.6

University of Alberta
Dept. of Civil Engineering
Triaxial Test on Compacted
Asphalt Stabilized Soil
Computations

Project Thesis

Sample No. 12

Length in. 1. 7.18 cm 2. 7.19 cm Av. 2.832 in

Diam. in. Top 1. 3.61 cm 2. 3.62 cm Av. 1.423 in. Area Top A_T 1.590 in^2

Centre 1. 3.61 cm 2. 3.61 cm Av. 1.421 in. Area Centre A_c 1.585 in^2

Bottom 1. 3.61 cm 2. 3.62 cm Av. 1.423 in. Area Bottom A_B 1.590 in^2

Average x-Sec. Area $= \frac{1}{4} (A_T + 2A_C + A_B)$

$=$ 1.588 in^2

Original Vol. $=$ 4.497 in^2

Vol. Change Upon Consolidation

Rdg. before consolidation .44

Rdg. after consolidation 4.20

Total vol. change 3.76 cc

Correction for cell expansion, etc. .80 cc

Vol. change in sample 2.96 cc = .18 in^3

Pore Pressure Response (for saturated samples only)

Incremental pressure = 20 psi

Str. indicator rdg. before application 2162

Str. indicator rdg. after application 331

Press response 18.7 psi

Per cent pore pressure response 94 %

Wt. of sample at: Beginning of test 124.88 gm

End of test 146.85 gm

Moisture Content at: Beginning of test —

End of test 19.4 %

<u>Degree of saturation</u>		<u>Void ratio</u>
Initial	6.71 %	.496
After consolidation	______	______
At failure	______	______
At end of test	111.37 %	.436

$\sigma_1' =$ 78.04 $\quad \sigma_3' =$ 12.7 $\quad \sigma_1 =$ ______ $\quad \sigma_3 =$ ______

$\frac{\sigma_1 + \sigma_3}{2} =$ 52.67 $\quad \frac{\sigma_1 + \sigma_3}{2} - u_w =$ 45.37

$\frac{\sigma_1 - \sigma_3}{2} =$ 32.67 $\quad \frac{\sigma_1 + \sigma_3}{2} - u_a =$ —

Sample Calculations for Triaxial Test Strength Values, Units Weights, Degree of Saturation and Void Ratio - Series SD, Sample No. 12

1. Triaxial Test Calculations

Strain $\epsilon = \dfrac{\text{change in height of sample}}{\text{original height of sample}}$

Load lbs. taken directly from the proving ring calibration chart

Corrected area $= \dfrac{\text{original area}}{1 - \epsilon}$

Deviator stress $\sigma_1 - \sigma_3 = \dfrac{\text{load lbs.}}{\text{corrected area}}$

Pore water pressure u_w , psi , taken directly from the transducer calibration chart

Effective lateral stress $\sigma_3' = \sigma_3 - u_w$

Effective normal stress $\sigma_1' = \sigma_3' + (\sigma_1 - \sigma_3)$

2. Unit Weight, Degree of Saturation and Void Ratio

Specific gravity of Braeburn Fines = 2.67
Specific gravity of asphalt $\approx$ 1.00
Sample wt. after compaction = 139.60
Moisture content after compaction = 13.2%

Dry wt. of sample $= \dfrac{139.60}{1.132} = 123.32$

Vol. of sample after compaction = 4.497 in^3

Total dry unit weight of sample $= \dfrac{123.32 \times 1728}{454 \times 4.497} = 104.35$ lbs/ft^3

Vol. of dry soil $= \dfrac{123.32}{1.042 \times 2.67} = 44.33$ cc

Vol. of asphalt $= 123.32 - \dfrac{123.32}{1.042} = 4.97$ cc

Vol. of solids $V_s = 44.33 + 4.97 = 49.30$ cc $= 3.007$ in^3

Vol. of voids $V_v = 4.497 - 3.007 = 1.490 \text{ in}^3$

Initial void ratio $e_o = \frac{V_v}{V_s} = \frac{1.490}{3.007} = .496$

Sample wt. after curing = 124.88 gm

Vol. of water in sample after curing = 124.88 - 123.32 =

1.56 cc =.10 in^3

Initial degree of saturation = $\frac{V_w}{V_v} = \frac{.10}{1.490} \times 100 = 6.71\%$

Vol. change due to consolidation = 2.96 cc = .18 in^3

Vol. of sample after consolidation = 4.497 - .18 = 4.317 in^3

Vol. of voids after consolidation = 4.317 - 3.007 = 1.310 in^3

Void ratio after consolidation = $\frac{V_v}{V_s} = \frac{1.310}{3.007} = .436$

Moisture content after testing = 19.4%

Vol. of water in sample after testing = .194 x 123.32

= 23.92 cc = 1.459 in^3

Degree of saturation after testing = $\frac{V_w}{V_v} = \frac{1.459}{1.310} \times 100 = 111.37\%$

APPENDIX D

CALCULATED PORE AIR PRESSURES

FOR SAND SAMPLES, SERIES UA

CALCULATED PORE AIR PRESSURE FOR
COMPACTED SAND SAMPLES

$$u_a = \frac{P_a \triangle V}{Va_o + hV_w - \triangle V}$$

Assume P_a = 13.56 psi (average atmospheric pressure for Edmonton area)

h = .02 (after Hilf, 1956)

Calculated u_a after consolidation

Sample No. 23 = .88 psi

Sample No. 24 = 1.53 psi

Sample No. 25 = 2.20 psi

Sample No.	Strain guage divisions	Vol. Change in^3	u_a psi Calculated	u_a psi Measured
23	0	0	.64	.27
	40	.027	1.32	.27
	80	.032	1.65	.43
	120	.040	1.95	.32
	160	.037	1.84	.26
	200	.031	1.61	.08
	240	.023	1.32	.02
	320	.005	.78	-.05
24	0	0	1.37	.35
	40	.030	2.23	.44
	80	.046	2.78	.42
	120	.055	3.04	
	200	.059	3.30	
	240	.058	3.27	.39
	320	.055	3.04	.30
25	0	0	1.51	.37
	40	.040	2.82	.33
	80	.066	4.30	.32
	200	.099	6.84	.29
	280	.103	7.18	.32
	320	.102	7.10	.27